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PETRONAS

WAVE FORCES ON FLOATING BREAKWATER

by

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Final Report submitted in partial fulfilment of
the requirements for the
Bachelor of Engineering (Hons)
(Civil Engineering)

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CERTIFICATION OF APPROVAL

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A project dissertation submitted to the
Civil Engineering Programme
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in partial fulfilment of the requirement for the
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Approved by,

(AP. Dr. Saied Saiedi)

UNIVERSITI TEKNOLOGI PETRONAS

TRONOH, PERAK

JUNE 2008

CERTIFICATION OF ORIGINALITY

This is to certify that I am responsible for the work submitted in this project, that the original work is my own except as specified in the references and acknowledgements, and that the original work contained herein have not been undertaken or done by unspecified sources or persons.

(NG BOON HOCK)

ABSTRACT

The main objective of this study is to further the studies made from previous experiences from Universiti Teknologi PETRONAS. These studies are to evaluate the wave forces that are applied to the floating breakwater. The report summarizes the laboratory tests conducted to evaluate the wave forces that are applied to the previously designed floating breakwater. Generally the geometry design of the floating breakwater will be the same, using different material. The laboratory tests were conducted in a controlled environment where tides and currents were not taken into account. The study includes a range of variation in terms of the environment that was tested. These variations include the difference of wave periods and water depth upon the floating breakwater. The water depths are 20cm and 30 cm. The tests showed that the peaked wave force occurs during the wave period of 1.35s. Also, it can be seen that the wave forces has a general relationship with the wave period, increasing as the wave period increases. A detailed study was therefore done to study the motions, the wave attenuation performance, and the energy distribution. It could be seen that at this particular wave period, the wave attenuation performance is rather low. The study of the motion shows that the floating breakwater was subjected to relatively uniform wave forces at this wave period. However, due to the vague methodology, a further detailed experimental setup should be done. The experimental study was done to compare with Goda's principle formula for breakwater design. However, it was observed that the formula was not suitable for floating breakwaters. An observation made from this study is that the wave forces induced were the highest for Model M-4, which is the inclined slope with keel plate, with the value of 9.58N.

Keywords: wave forces, floating breakwater.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF SYMBOLS

A	wetted surface area
D	draft
d	water depth
C_t	coefficient of transmission
C_R	coefficient of reflection
E	wave energy
E_p	wave energy due to displacement of the water surface
E_k	wave energy due to water particles associated with wave motion
EI	cross sectional properties of the pile
f	frequency
g	gravity acceleration
H	wave height
k	wave number
L	wavelength
L_0	deepwater wavelength
M	moment induced by wave forces
P	average energy flux per unit wave crest width
p	pressure intensity induced by wave
T	wave period
U	internal strain energy
x	displacement
GB	distance between centroid and the center of buoyancy
MB	righting moment from the center of buoyancy
MG	metacentric height
α_1	coefficient that takes into account of wave period to the pressure intensity
α_2	coefficient that takes into account the presence of rubble mound foundation to the pressure intensity
α_3	coefficient derived by relation of linear pressure distribution
β	wave angle
γ	specific weight of water
ρ	water density

CHAPTER 1

INTRODUCTION

1.1 Problem Statement

Erosion has bee a problem in the discipline of coastal engineering. It was reported by the National Erosion Study that was conducted by the Malaysian government that about 1,300 km of Malaysia's total coastline is subjected to erosion. In most cases, erosion occurs due to natural forces. However, erosion can occur due to interference by men to nature as well.

Coastal erosion can be divided into three different categories, i.e. critical erosion areas, significant erosion areas, and acceptable erosion areas depending on the economic value of the effected properties as shown in Table 1.

Table 1.1 Summary of eroding coastline in Malaysia

Category	Description	Total length (km)	Percentage
1	Critical Area where facilities are in irradiate danger	140	11%
2	Significant Area where facilities will become endangered in 5 to 10 years time	240	19%
3	Acceptable Are which generally undeveloped and has no facilities	900	70%
Total		**1,280**	**100%**

As Malaysia is a developing country, coastal erosion would cause a lot of damage and losses. Coastal erosion has resulted in damages and loss of agriculture land, mangrove forests, housing facilities, roads, beaches, etc. These would amount to many millions of Ringgit, and facilities and properties that are subjected to severe erosion may soon be endangered. The concern about the economical and social consequences of the coastal erosion has been increased due to the intensified development of the country.

Based on this, the control of costal erosion has become an economic and social need. An immediate solution to this is to protect the facilities and property of the eroding area using breakwaters.

Fixed breakwaters are all built to attenuate waves, reducing the wave energy by the means of reflection. However, these bottom-founded structures are impossible in deepwater conditions from both technical and economic point of view. In previous reports (d' Angremond et. al.,1998), it was already stated that the conventional breakwater, from an economic point of view, will only be preferable until a water depth of around 8m. In the technical point of view, a fixed breakwater is not feasible in deepwater conditions. It is known that the deeper it is, the larger would be in the hydrostatic forces. Therefore with high waves, when the critical load is exceeded, the structure will lose stability at once. Also, when oncoming waves hit these breakwaters, their erosive power is concentrated on these structures some distance away from the coast. In this way, there is an area of slack water behind the breakwaters. Deposition occurs in these waters and beaches can be built up or extend in these waters, forming a tombolo. However, nearby unprotected sections of the beaches do not receive fresh supplies of eroded sediments and may gradually shrink due to erosion, as shown in Figure 1.1. An additional negative aspect is that for a fixed breakwater, once it is fixed, it is rarely moved due to its cost. A fixed breakwater needs to be carefully designed and analyzed for its effects towards the environment. Fixed breakwaters close a significant portion of the waterway, causing a faster river or tidal flow. Moreover there is a potential to trap debris on the updrift side of the breakwater. The negative impacts of fixed breakwaters can be summarized in the table below:

Table 1.2 Negative impacts of fixed breakwaters

No.	Description
1	Once fixed, it is rarely removed (expensive). Needs to be carefully designed and analyzed for its effects towards the environment.
2	Close of a significant portion of the waterway. Causing a faster river or tidal flow.
3	Potentially trapping debris on the updrift side.
4	Water quality problems due to poor water circulation behind the structure.
5	May form a tombolo, interrupting the longshore transport and cause downward erosion.
6	Dampening power decreases when tide level increases, because wave dissipation of the wave over the breakwater is caused by wave breaking on the slope. Fixed breakwaters are considered most economical if the water level is at about 4 feet (McCartney, 1985).

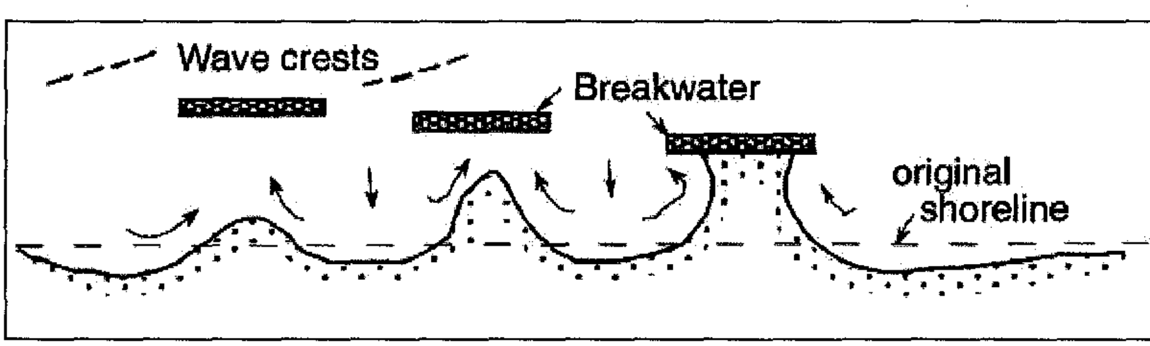


Figure 1.1 Effects of a fixed floating breakwater (http://wikipedia.org)

Coastal engineering projects often have a significant effect on natural ecosystems and may contribute to some certain permanent damage for the future generation. Due to the negative impacts mentioned above, coastal engineers have moved on to a softer approach, moving towards working with the environment, rather than going against it. From there, floating breakwater was revolutionized and appeared to be a cost-effective substitute for the conventional breakwaters.

1.2 Objective of the Study

a) The primary objective of this study is to further the study made by previous experiences from Universiti Teknologi PETRONAS.

b) To evaluate the wave forces that will be applied on a floating breakwater.

c) Also, wave forces that will be applied on various designs of floating breakwater (different geometries of previous designs) will be evaluated, noting the differences of performance between different designs and materials.

CHAPTER 2

THEORY

2.1 Literature Review

2.1.1 Breakwaters in General

One method of coastline protection is to build breakwaters at the applicable location. Breakwater is widely used around the world. The main function of a breakwater is to attenuate wave, in other words, to reduce or eliminate the wave energy in inshore waters, with this, reducing the erosion of a beach. A breakwater can be used as part of coastal defense or to protect and anchorage from the effects of weather and alongshore drift. It is used to reduce the intensity of wave action in the inshore waters therefore reduce coastal erosion. It is also used to protect vessels in port and for port facilities. Moreover, a breakwater can be used to protect valuable habitats that are threatened by the destructive forces of the sea as well as prevent or reduce the siltation of navigation channels. Also, a breakwater can accommodate loading facilities for cargo or passengers.

Traditional breakwaters are fixed solid structures. Breakwaters are substantial structures, located at the outer limits of a harbor or anchorage, to protect the inner water against the effects of heavy seas and winds and to ensure safe mooring, operating, loading, or unloading of shipping within the harbor. Figure 2.1 shows a typical placement of a breakwater. One of the most conventional breakwaters is the rubble-mound breakwater. This type of breakwater is typically constructed with a core of quarry-run stone, sand or slag, and protected one or more stone under layers and a cover layer composed of stone or concrete.

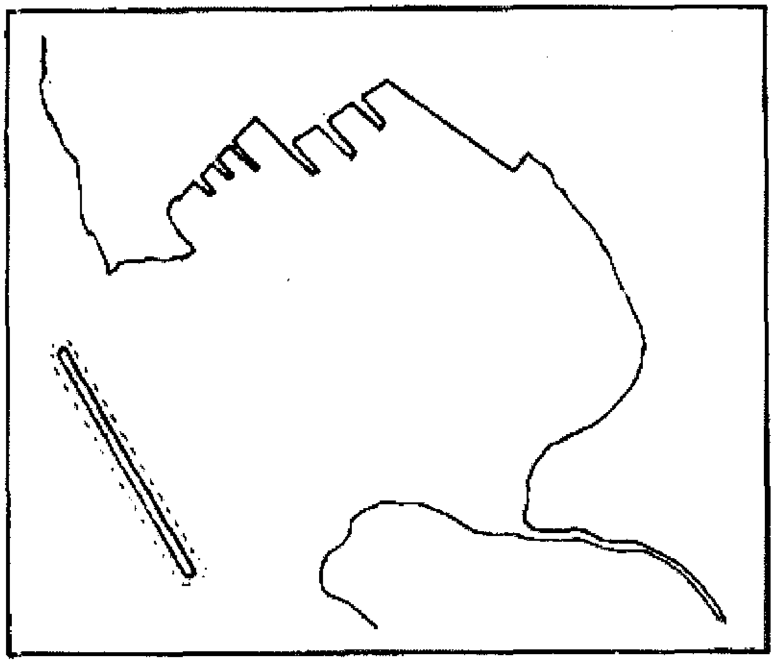


Figure 2.1 A typical placement of a breakwater

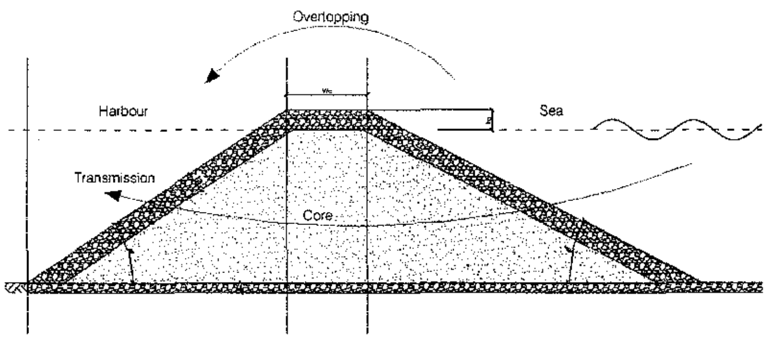
Besides those that were mention above, there are other types of fixed breakwaters. The difference types of fixed breakwaters depend on the normal water depth and tidal range. The fixed breakwaters can be categorized based on their structural features. M.W. Foustert (2006) distinguishes three main types of breakwater, which are,

a) Conventional (rubble-mound) type breakwaters (Figure 2.2a)
   The breakwater is fixed, pervious gravity structure constructed of graded rock materials, with one or more stone underlayers and a cover layer composed of stone or specially shaped concrete armor. This is used to minimize the transmission and overtopping wave energy. This type of breakwater is usually have varying slope profile, but no lesser than 1 to 10 (Teh, 2002). This type of breakwater uses the voids in the structure to dissipate energy. Also, the rock or concrete armor is used to absorb energy.

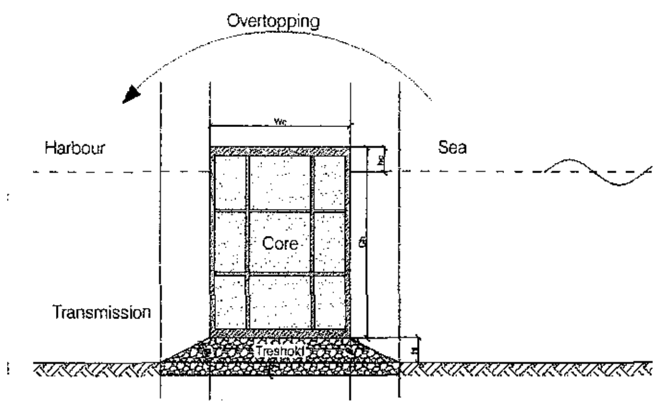
b) Monolithic type breakwaters (Figure 2.2b)
   Has a cross section designed in such a way that the structure acts as one solid block. This type of breakwater uses its mass to resist the overturning forces, much like a dam.

c) Composite breakwaters (Figure 2.2c)

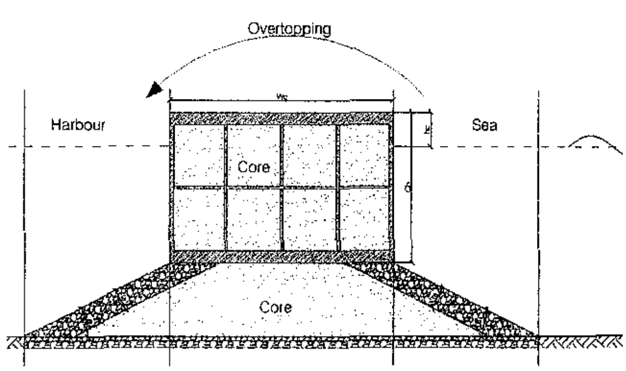
It is a combination of both conventional and monolithic type of breakwater. This type of structure is preferred when the water depths get large.



(a) Conventional Type



(b) Monolithic type



(c) Composite type

Figure 2.2 Several types of breakwaters

Although the designs mentioned above have physical difference, these structures have a lot of similarities. These structures are all built to attenuate waves, reducing the wave energy. However, these bottom-founded structures are impossible in deepwater conditions from both technical and economic point of view. In previous reports (d' Angremond et. al.,1998), it was already stated that the conventional breakwater, from an economic point of view, will only be preferable until a water depth of around 8m. In depths ranging from 8m to 20m, a caisson breakwater will be the best solution. And after that, up to a depth of 30m, the composite type of breakwater is preferable. The monolithic-type require a solid foundation that can cope with high loads. When the critical load is exceeded, the structure will lose stability at once. Also, when oncoming waves hit these breakwaters, their erosive power is concentrated on these structures some distance away from the coast. In this way, there is an area of slack water behind the breakwaters. Deposition occurs in these waters and beaches can be built up or extend in these waters, forming a tombolo. However, nearby unprotected sections of the beaches do not receive fresh supplies of eroded sediments and may gradually shrink due to erosion, as shown in Figure 1.1. An additional negative aspect is that for a fixed breakwater, once it is fixed, it is rarely moved due to its cost. A fixed breakwater needs to be carefully designed and analyzed for its effects towards the environment. Fixed breakwaters close a significant portion of the waterway, causing a faster river or tidal flow. Moreover there is a potential to trap debris on the updrift side of the breakwater.

2.1.2 Floating Breakwaters in General

Coastal engineering projects often would have a significant effect on the environment and the ecosystem. These negative effects would worsen if not remedial work is being done and would make things worst for future generations. Even though fixed breakwaters provide a higher degree of protection, it cannot compete with the floating breakwater cost wise, especially in deep water conditions. A fixed breakwater may not be competitive cost wise with a floating breakwater (depending on the incident wave period) at depths greater than about 10 feet (Hales, 1981). In recent years, engineers have been more environmental conscious. They are coming up with soften approach to attenuate wave energies, for a structure to perform like a fixed breakwater, minus the harm that would be caused by it. This therefore brings up a new approach; the floating breakwater has proved to be a cost-effective substitute for the conventional fixed breakwater, providing the adequate protection while working with the environment. A cross section view of a typical floating breakwater is shown in Figure 2.3.

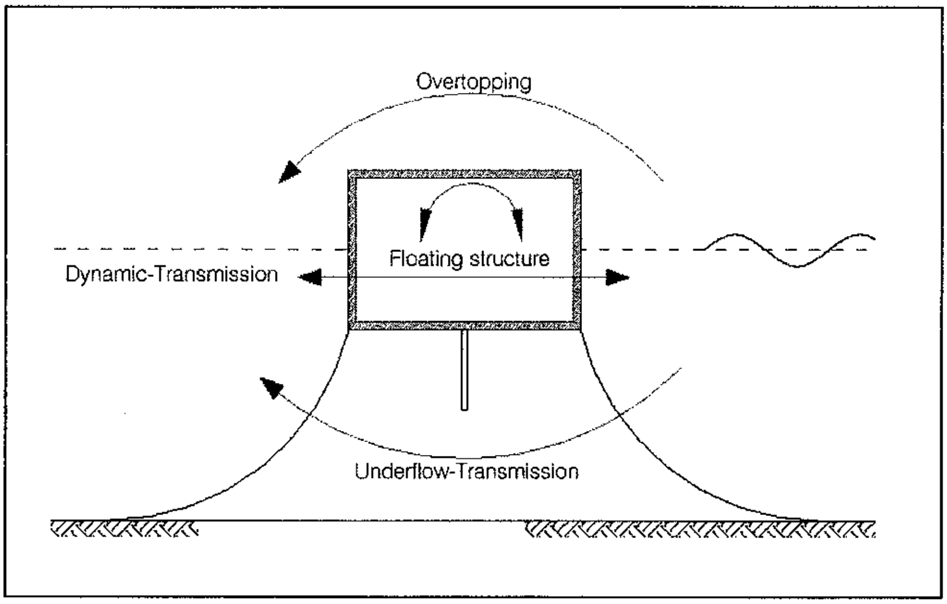


Figure 2.3 Cross sectional view of a typical floating breakwater

A floating breakwater is a breakwater that is constructed to posses a limited range of movement. It has the basic functions of a breakwater, but having less attenuating properties compared to the fixed breakwater. Studies were performed on the use of floating breakwaters to the protection of coastline erosion, and other important natural and man-made marine structures from the effects of wave. From these studies, there are many different types of floating breakwaters have been developed and many conclusions were made as to their effectiveness. The studies can therefore be categorized into two (Foustert, 2006).

2.1.2.1 Construction

Cost is not the only problem that renders conventional breakwaters to be non-preferable in deep water conditions. The soil conditions and structural stability also play a major role in the influence of the limits of a fixed breakwater design. A huge structure would cause large pressure on the subsoil. Therefore, a floating breakwater would be more feasible in poor soil conditions compared to heavy conventional breakwaters.

The construction period of a floating breakwater is evidently much shorter compared to a conventional breakwater. Unlike a conventional breakwater, floating breakwaters can be easily fabricated on site or at factories. Also, floating breakwaters are not as obstructive as conventional breakwaters and it can be aesthetically pleasing (McCartney, 1985).

2.1.2.2 Performance

A conventional fixed breakwater has better wave attenuating wave abilities compared to a floating breakwater. Floating breakwaters can only attenuate wave heights at a limited frequency range. The floating breakwater would have a simultaneous movement with a certain wave frequency. It is also adaptable to tidal changes because a floating breakwater it freely floats on the water surface, and not subjected to any fix support.

Floating breakwaters are 'environmentally friendly'. They do not extend all the way to the bottom of the seabed, therefore they have minimal interference with water circulation, littoral transport, sediment transport, shore processes, and fish migration. And floating breakwaters are susceptible to any sudden structural failure during catastrophic storms. Floating breakwater can act as a multi-purpose breakwater facility as well, permitting greater multi-use potential than fixed conventional breakwaters. Besides working as a wave attenuator, a floating breakwater can function as a walkway, marine habitat, pier and dock boat too (Lim, 2006).

A summary of the technical arguments can be listed below.

a) A huge structure (i.e. conventional breakwater) will result in tremendous pressure on the subsoil as well as stability problems when the slope becomes too deep.

b) Floating breakwaters have minimal interference with water circulation, sediment transport and fish migration.
c) Floating breakwaters can be easily moved and rearranged due to their transportability, reusability, and flexibility in design.
d) Floating breakwaters are susceptible to a sudden structural failure during catastrophic storms. However, if the mooring fails, a floating breakwater can be a threat for the harbor facilities.

Nonetheless, floating breakwater pose some negative impacts which requires careful considerations in their evaluation. As mentioned above, a major disadvantage is that the frequency range of wave heights that a floating breakwater can attenuate is limited. Therefore careful and precise engineering is needed to research, design and development of a floating breakwater. The floating breakwater must be carefully match specifications of a site (Lim, 2006). Also, there are other drawbacks when using a floating breakwater, such as the design life of a floating breakwater is very much shorter (10 – 20 years) compared to a fixed breakwater (100 years). Higher maintenance is needed for a floating breakwater, such as the maintenance of the corrosion problem of the mooring line. Hales (1981) stated that the uncertainties in the magnitude and types of applied loading on the system, and lack of maintenance cost information, discrete conservative design practices which naturally increases the initial project cost. As mentioned above, the floating breakwater has a limited movement that goes along with the wave frequency. This would render the floating breakwater to be prone to fatigue.

2.1.2.3 Types

The different types of floating breakwaters may be seen as a combination of variation of materials, shape, mooring system, and function. Floating breakwaters generally can be categorized into four main types based on their geometric and functional similarities, namely,

a) Box
   i. Made of reinforced concrete

ii. Rectangular in cross section shaped models that either flexibly or rigidly fixed with other models
iii. Advantage: recreational, boat mooring
iv. Disadvantage: expensive and high maintenance

b) Pontoon

i. It has prismatic shape
ii. Advantage: can function as floating walkway, storage, boat moorings, and fishing piers
iii. Similar advantages and disadvantages as box type floating breakwater

c) Tire mat

i. Binding a group of tires together
ii. Advantage: low cost, simple design and construction, portability, low anchor loads, and greater effectiveness.
iii. Disadvantage: low buoyancy, low design life, not effective against long wave lengths, cannot be moored year round due to icing effects, if not properly bind can break apart and be a hazard.

d) Tethered (moored) floating breakwater

i. Unlike those above that uses mass to attenuate waves, this uses the mooring system.
    - Wave moves around the breakwater until the mooring system restricts the motions.
    - Then the waves are transferred to the anchor and ultimately the seafloor, dissipating the wave height.

The concepts of a floating breakwater have been developing since World War 2. Many papers have been written to serve to collect and review the progress of this development. These papers include the works of O'Brian, Kuchenreuther and Jones (1961), Jones (1971), Griffin (1972), Richey and Nece (1972, 1974). Adee (1975) has written a paper in an attempt to collect information on floating breakwater which have already been designed and built. This paper also compares the few prominent floating breakwaters that have already been constructed.

2.2 Floating Breakwater Applicability and Advantages

Floating breakwaters are usually applicable when there are environmental and financial restrictions on marina and other coastal facilities. In a place where the coastal area needs shore protection, an alternative to a conventional breakwater which has many restrictions in the both operational and financial sense would be a floating breakwater. Unlike a conventional breakwater that only utilizes it's mass to reflect wave energy, a floating breakwater is a concept that uses reflection, dissipation of energy and/or transformation to reduce wave energy to an acceptable level (Morey, 1998).

A floating breakwater is a breakwater that relies on the draft for wave-structure interaction in the upper portion of the wave. Generally, a floating breakwater consists of a floating pontoon that is hold down from drifting away by cables or chains that connect it to the seabed. Floating breakwaters have a general function of protecting the lee side of it by reducing wave energies. They are usually installed at shores which need coastal protection (i.e. coastal erosion), marina for boat docking, and recreational area for the public.

There are many advantages of using the floating breakwater instead of a conventional breakwater, especially when it comes down to financial restrictions. This is because a floating breakwater has lower capital cost and shorter construction time. According to Hales (1981), a floating breakwater is much more cost effective and is economical alternatives, especially in deep water regions. In water depth greater than about 3.05m, a conventional breakwater is not competitive cost-wise compared to a floating breakwater. It is known that the wave forces only dissipate over the upper portion of the breakwater, rendering the rest of it ineffective. Therefore it is much more economical to attenuate wave energy only in the vicinity of the free surface. In addition, a floating breakwater is more flexible compared to a fixed breakwater relative to the water depth as well as the variety of seabed conditions. As it does not extend to the full depth of water, this means that a floating breakwater is very much independent from the seabed, whether it is poor soil foundation or varying seabed profile, it is very versatile. While the construction cost of a conventional breakwater increases with water depth, the cost of a floating breakwater is much less sensitive to depth and the seabed conditions. Also, the construction period of a floating breakwater is less compared to a conventional breakwater, reducing the construction cost as well.

Other advantages of floating breakwaters is that they are mobile, and they can be relocated or rearranged based on the best layout for the optimum wave attenuation performance based on seasonal patterns. Also, floating breakwaters are adaptable to water level change, meaning that the performance are not subjected to tidal level as they are freely floating on the water surface. All in all, a floating breakwater can be realigned with minimum effort as it is facilitate by the buoyant and mobile characteristics of the structure.

Floating breakwaters are 'environmental friendly' as well. Because the floating breakwater does not extend all the way down to full depth, this allow free circulation of the water and do not form barrier for suspended and bedload sediment transport. The open portion of the floating breakwater at the bottom also allows seawater exchange that is essential to fish migration. Apart from these, floating breakwaters can be aesthetically pleasing as it has a low profile and present minimum intrusion on the horizon, particularly at high tide ranges areas (McCartney, 1985).

2.3 UTP Experience "Wave Suppress System"

Teh et al. (2006) has developed the Wave Suppress System (WSS) to arrest the destructive power of water waves. Based on this paper, it was reported that one of the factors to ensure better wave attenuation performance is through broadening the floating breakwater's width to cover a substantial faction of the length of waves (Cox et al., 1991). However, this is objectionable as the spread may lead to higher costs as well, difficulty in installation and handling, and it pose danger to navigation.

Teh et al. (2006) has developed a limited width floating breakwater, which is the first generation of WSS (GEN-1). This model provides a relatively satisfying wave attenuation performance. This model is a floating breakwater that has a zigzag shape and sharp edge features that would effectively attenuate waves impinging on the structure (Figure 2.4). It is made of impermeable lightweight concrete and tied with a mooring system, with the aim to provide the required level of wave tranquility in an area it is desired to protect. The model has two arms on the top of the rectangular body, and two legs at the bottom. The function of the two arms is to facilitate wave breaking, whereas the two legs at the bottom are to enhance the stability of the model.

In this paper, Teh et al. (2006) has further improved the characteristics of the GEN-1 model. Two major enhancements was made for further improvement to the GEN-1 model, which is to alter the bottoms legs of the model into 45°-slanting legs; and keel plates were connected to beneath of the model to form a V-shape configuration. This model was later named the GEN-2 model (Figure 2.5and Figure 2.6). The properties of the three tested WSS models are briefly summarized in Table 2.1.

Table 2.1 Properties of the WSS models

WSS model	Density (kg/m^3)	Freeboard (cm)	Draft (cm)	Materials	Side slope (degree)
GEN -1	650	3.5	6.5	Autoclaved Lightweight Concrete (ALC) with fiberglass coating	90
GEN-2	545	5.1	4.9	ALC with waterproofing emulsion	45
GEN-2 with keel plate	556	5.0	10.3	ALC with waterproofing emulsion with zinc sheet	45

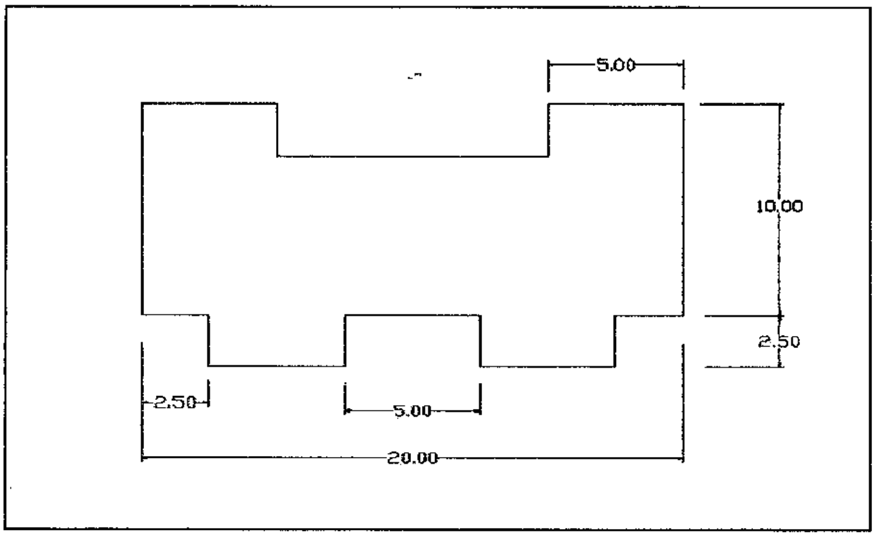


Figure 2.4 Cross sectional view of the GEN-1 model

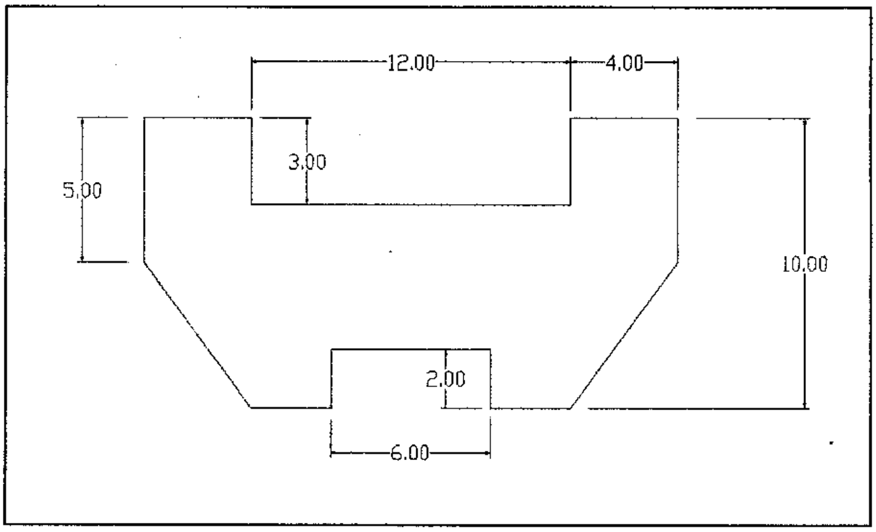


Figure 2.5 Cross sectional view of the GEN-2 model

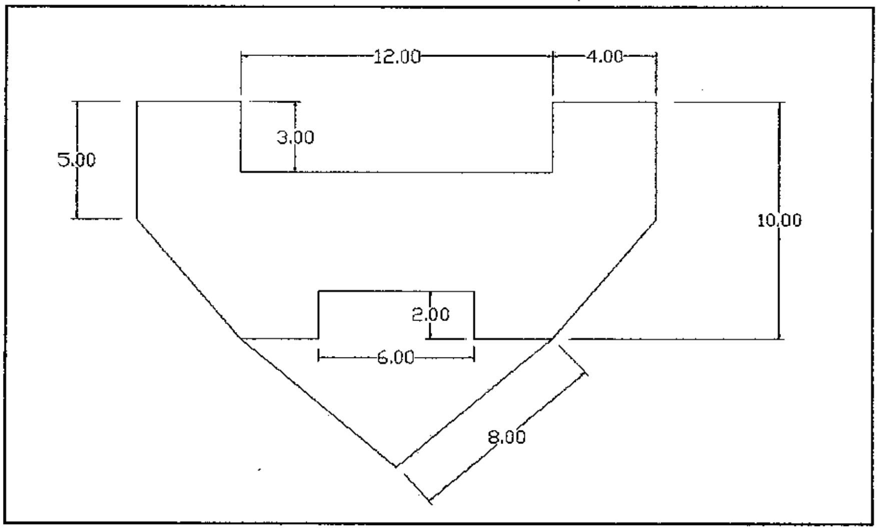


Figure 2.6 Cross sectional view of the GEN-2 model with keel plates

The experiments done for the WSS models were conducted in a controlled environment where tides and currents were not taken into account. These experiments were done in the wave flume with dimension of 12m long, 0.3 wide, and 0.45 deep of Universiti Teknologi PETRONAS. The flume was subjected to monochromatic waves with a wave dampener at the end of the flume to prevent any reflective from affecting the experiments. Also, the waves were conducted for various wave periods ranging from 0.5 seconds to 2.0 seconds in 20cm, 25cm, and 30cm water depths. Overall, there were more than 300 tests that were conducted. All the models were tested with limited width.

Generally the adopted criteria to evaluate the performance of the WSS models are the reflection coefficients, energy lost coefficient, and transmission coefficient.

In general, the tests show that the GEN-1 model is an effective wave-energy dissipater but a poor wave reflector. The GEN-1 model attenuates the incident waves mainly by the mechanism of dissipation through breaking and friction over body of the floating structure, rather than by reflection effect. Also, the tests show that the degree of wave attenuation was dependent strongly upon the geometric factors, namely the B/L and wave conditions, H^i/gT^2. In addition to that, the transmission coefficient was higher when the model was exposed to longer period waves. Also, the GEN-1 model is concluded to perform better in shallow waters.

The effect of wave reflection becomes more appreciable when the models, especially the GEN-2 model and GEN-2 with keep plate model are exposed to waves with high steepness and in limiting depth of water. The models' energy loss performance was very encouraging as the energy loss coefficient, C_L of the respective models were found to be greater than 0.5. Not only that, the coefficient shows that C_L increases with Hi/gT^2. In addition to that, the wave attenuation performance for all models improves with increasing Hi/gT^2.

The enhanced model (i.e. GEN-2 and GEN-2 with keep plate) improved the GEN-1's poor reflective performance. The modification made to the slope and with the addition of the keep plate made these models a better wave reflector. The keel plates are added to the configuration to allow water within the plates to be trapped sufficiently, acting as ballast and assisting in retarding the heave movement of the module in response to the wave action. The tests therefore showed that with the addition of the keel plate, the reflective performance is considerably enhanced making this model superior in dissipating wave energy through reflection and energy dissipation effects.

2.4 Waves and Wave Forces

Ocean surface waves, better known as waves, are surface waves that occur in the upper layer of the ocean. They are mechanical waves that propagate along the interface between water and air. Waves occur from wind or geologic effects and may travel a distance before reaching the shore. They range in size from small ripples to huge tsunamis. In an individual wave, there is little actual forward motion of individual water particles, despite the large amount of energy and momentum it may carry forward. Waves can also be defined by the fluctuation of the water level, accompanied by a local current, accelerations and pressure fluctuation. The simplest form of a wave is sinusoidal, but in actual it is very complex.

The formation of waves is due to the disturbance caused by energy moving through water mass. The disturbing forces that cause waves are due to wind, displacements (earthquakes, landslides, tsunami, etc.), changes in atmospheric pressure, and the gravitational pull of the sun and moon. However, the majority of waves that can be seen by the beach results from distant winds.

2.4.1 Waves Characteristics

Waves in general have a range of sizes. The size of the wave is generally measured by the height of the wave (from crest to trough), or the period of the wave (as shown in Figure 2.7). For most of coastal engineering that involves wave statistics, their size over a period of time is usually expressed as "significant wave height." This figure represents the average height of the highest one-third of the waves in a given time period or in a specific wave or storm system.

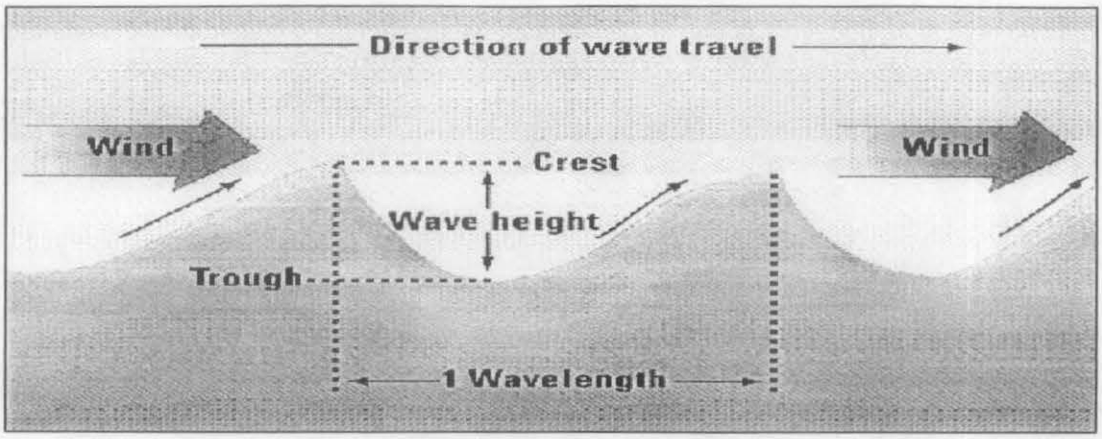


Figure 2.7 Wave profile

The wave process is as the following: As the wind blows, pressure and friction forces disturb the equilibrium of the ocean surface. These forces transfer energy from the air to the water, forming waves. In the case of deep water waves, particles near the surface move in circular paths, making ocean surface waves a combination of longitudinal and transverse wave motions. When waves propagate in shallow water, the particle trajectories are converted into ellipses (Figure 2.8). As the wave amplitude increases, the particle paths no longer form closed orbits. After the passage of each crest, particles are displaced a little forward from their previous positions, a phenomenon known as Stokes drift. Stokes drift can be defined as the drift of particles in gravity waves, which arises from the fact that particle velocities are periodic with a mean which is not zero.

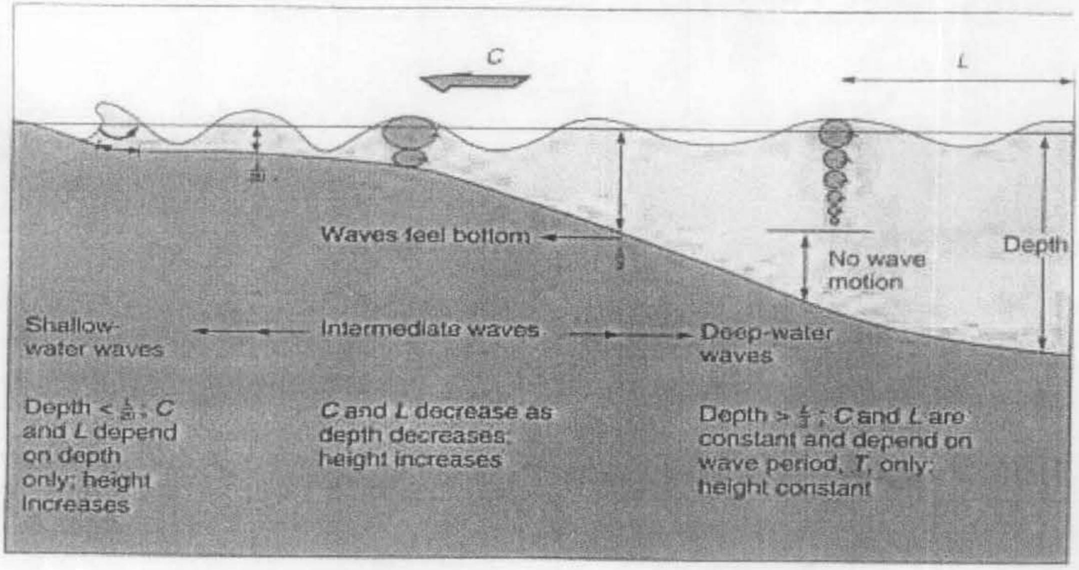


Figure 2.8 Motion of a particle in a wave

As mentioned above, waves are generated by wind passing over the sea. This would exert a lateral stress (in the form of friction) on the water's surface, resulting in the formation and growth of waves. Then the turbulent air flowing close to the water surface creates further rapidly changing sheer stresses and pressure fluctuations into energy and finally when waves have reached a certain size the wind can exert a stronger force on the upwind face of the wave causing additional growth.

This energy density (per unit area) of regular sinusoidal waves depends on the water density, ρ, gravity acceleration, *g* and the wave height, *H*. The wave energy is obtained by summing the energy due to the displacement of the water surface and the energy due to water particles associated with wave motion (Teh, 2007). The wave energy therefore can be defined as such.

$$E = E_P + E_k = \frac{1}{16}\rho g H^2 + \frac{1}{16}\rho g H^2 = \frac{1}{8}\rho g H^2 \qquad (2.1)$$

where E_P is the energy due to the displacement of the water surface, and
E_k is the energy due to water particles associated with wave motion.

From the equation above, the wave energy can be obtained with the knowledge of the wave height. Along with the wave celerity, the wave energy flux, better known as the wave power, can be obtained. Wave power is the rate at which energy is transmitted in the direction of the wave propagation across vertical plane perpendicular to the direction of wave advance and extending down the entire depth (Teh, 2007). The wave power can be defined as such.

$$P = E \times C_G \qquad (2.2)$$

Where P = the average energy flux per unit wave crest width
E = total wave energy
C_G = the group velocity

The wave theory that is applicable in this study is the Linear Airy Theory. A summary of the equations obtained from this theory is shown in Table 2.2:

Table 2.2 A summary of Linear Airy Theory

Phase $\theta = kx - \omega t$ Relative water depth d/L	Deep water d/L > 0.5	Finite water depth d/L < 0.5
Velocity potential Φ	$\frac{gH}{2\omega} e^{ky} Sin\theta$	$\frac{gH}{2\omega}\frac{\cosh ks}{\cosh kd} Sin\theta$
Surface elevation η	$\frac{H}{2} Cos\theta$	$\frac{H}{2} Cos\theta$
Dynamic pressure $-\rho\frac{\partial\phi}{\partial t}$	$\rho g \frac{H}{2} e^{ky} Cos\theta$	$\rho g \frac{H}{2}\frac{\cosh ks}{\cosh kd} Cos\theta$
Water particle velocities horizontal $u = \frac{\partial\phi}{\partial x}$	$\omega\frac{H}{2} e^{ky} Cos\theta$	$\omega\frac{H}{2}\frac{\cosh ks}{\sinh kd} Cos\theta$
vertical $v = \frac{\partial\phi}{\partial y}$	$\omega\frac{H}{2} e^{ky} Sin\theta$	$\omega\frac{H}{2}\frac{\sinh ks}{\sinh kd} Sin\theta$
Water particle accelerations horizontal $\dot{u} = \frac{\partial u}{\partial t}$	$\omega^2 \frac{H}{2} e^{ky} Sin\theta$	$\omega^2 \frac{H}{2}\frac{\cosh ks}{\sinh kd} Sin\theta$
vertical $\dot{v} = \frac{\partial v}{\partial t}$	$\omega^2 \frac{H}{2} e^{ky} Cos\theta$	$\omega^2 \frac{H}{2}\frac{\sinh ks}{\sinh kd} Cos\theta$
Wave celerity $c = \frac{\omega}{k} = \frac{L}{T}$	$c_0 = \sqrt{\frac{g}{k_0}} = \frac{g}{\omega}$	$c = \sqrt{\frac{g\tanh kd}{k}}$
Group velocity c_{gr}	$c_{gr_0} = \frac{c_0}{2} = \frac{g}{2\omega}$	$c_{gr} = \frac{c}{2}\left[1 + \frac{2kd}{\sinh 2kd}\right]$
Circular frequency $\omega = \frac{2\pi}{T}$	$\omega_0 = \sqrt{gk}$	$\sqrt{gk\tanh kd}$
Wave length $L = \frac{2\pi}{k}$	$L_0 = \frac{gT^2}{2\pi}$	$L = \frac{gT^2}{2\pi}\tanh kd$
Wave number $k = \frac{2\pi}{L}$	$k_0 = \frac{\omega^2}{g}$	$k = \frac{\omega^2}{g\tanh kd}$
Water particle displacements horizontal ξ	Circular orbits $-\frac{H}{2} e^{ky} Sin\theta$	Elliptical orbits $-\frac{H}{2}\frac{\cosh ks}{\sinh kd} Sin\theta$
vertical η	$\frac{H}{2} e^{ky} Cos\theta$	$\frac{H}{2}\frac{\sinh ks}{\sinh kd} Cos\theta$
Energy Density E (J/m^2) Wave Power (W/m)	$\rho g H^2/8$ Ec_{gr}	$\rho g H^2/8$ Ec_{gr}

2.5 Goda's Principles for Breakwater Design

The author of these principles is Yoshimi Goda. He is from the Department of Civil Engineering Yokohama National University in Japan. This paper was published for the International Conference on Coastal Engineering (ICCE) in 1992 by *Instituto di Idraulica, Universita di Bologna,* Italy.

In this paper, various pressure formulations are discussed and the designed formulae employed in Japan are discussed with sample calculations. Also, there are several design factors that were discussed.

The paper is mainly about designing upright breakwaters. These breakwaters are the conventional type, which is the have a foundation at the seabed. In this paper, the development of the breakwaters is mainly in Japan, where the design formulas are applied in mainly in Japan.

The paper has some reviewed on the historical development and the features of Japanese breakwaters. But the main points in the paper are the formulas derived and will be applied in the breakwater design. Along this line, the paper reviewed about the wave pressure and forces applied on the vertical wall of the breakwater. These wave forces and pressure formulas include Hiroi's formula, Minikin's formula and others. With these formulas, the design formulae of wave pressure were analyzed. Other wave parameters, such as the design wave, the wave period, and the angle of wave incidence, wave buoyancy, and uplift pressure were discussed as well. Finally, other design factors were discussed and a calculation example was given.

The section that this study is interested from this paper is the design formulae of wave pressures for upright breakwater. In this section, there is a proposed universal wave pressure formula that would be useful for theoretical calculation of this study.

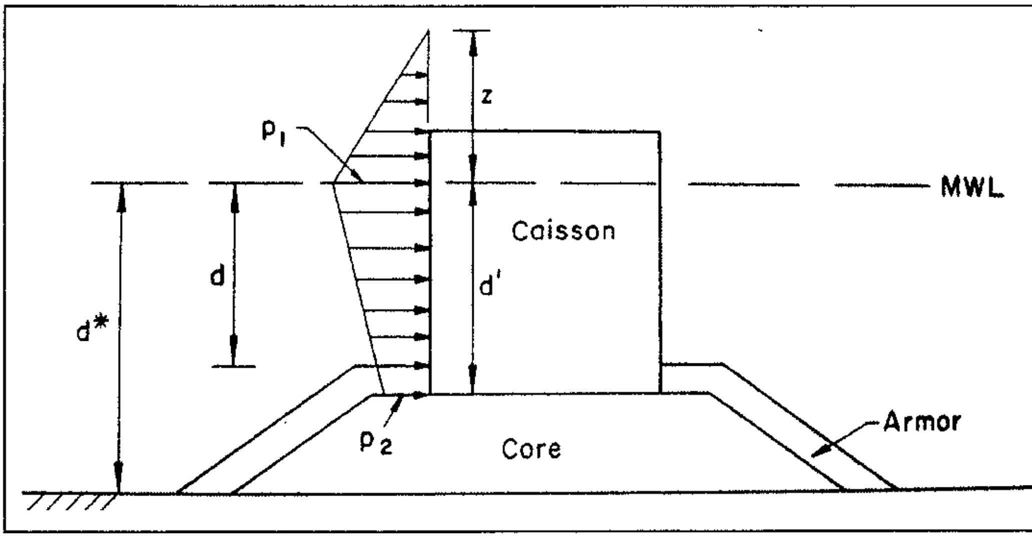


Figure 2.9 Wave pressure distributions by Goda's Formula

The wave distribution presented by Goda is as above in Figure 2.9. Like most hydro related forces, the pressure distribution has the largest intensity at the design water level and it decreases linearly towards the seabed or the bottom of the structure. The largest pressure intensity in this case is designated as p_1 and the pressure intensity at the bottom is designated as p_2.

In this study, because the structure is a floating body, the structure will be constantly placed at the designed water level. Therefore, this study will take the largest pressure intensity, which is the pressure intensity that is applied to the top part of the structure in Figure 2.9, which is at design water level, as the theoretical wave pressure distribution. The intensity of p_1 can be calculated by the following:

$$p_1 = 0.5(1+\cos\beta)(\alpha_1 + \alpha_2 + \cos^2\beta)\gamma H_{\max} \quad (2.3)$$

$$p_2 = \alpha_3 p_1 \quad (2.4)$$

$$\alpha_1 = 0.6 + 0.5\left(\frac{2kd}{\sinh 2kd}\right)^2 \quad (2.5)$$

$$\alpha_2 = {}^{2d}\!/_{H_{\max}} \quad (2.6)$$

$$\alpha_3 = 1 - {}^{D}\!/_{d}\left[1 - {}^{1}\!/_{\cosh kd}\right] \quad (2.7)$$

Where β is the wave angle, γ is the specific weight of water, H is the wave height, k is the wave number, d is the water depth, D is the draft, and α_1 and α_2 are coefficients.

Theoretically, the force can be obtained by multiplying the pressure distribution with the surface area of the previously designed floating breakwater by Universiti Teknologi PETRONAS. This force would be the theoretical total force applied to the floating body. This value will then be the compared with the experimental values of this study for further evaluation.

The coefficient α_1 and α_2 has respective representations. The coefficient α_1 represents the effect of wave period on the wave pressure intensities. The coefficient will have a minimum value of 0.6 for deepwater waves and a maximum value of 1.1 for very shallow water waves. The coefficient α_2 is introduced into the equation to take into account the presence of a rubble mound foundation. The coefficient α_3 is derived by the relation of linear pressure distribution. These coefficients were empirically defined through laboratory experiments on wave pressures.

Since the coefficient α_2 was introduced into the equation to take into account the presence of a rubble mound foundation, the wave pressure intensity formula is modified to remove the coefficient α_2. This is because in the case of this study, this is a floating body and therefore there is no rubble mound foundation. The equation is modified from eq. 2.3, and thus:

$$p_1 = 0.5(1+\cos\beta)(\alpha_1 + \cos^2\beta)\gamma H_{\max} \qquad (2.8)$$

$$p_2 = \alpha_3 p_1 \qquad (2.9)$$

$$\alpha_1 = 0.6 + 0.5\left(\frac{2kh}{\sinh 2kh}\right)^2 \qquad (2.10)$$

$$\alpha_3 = 1 - {d'}/{d}\left[1 - {1}/{\cosh kd}\right] \qquad (2.11)$$

$$p = {p_1 + p_2}/{2} \qquad (2.12)$$

where the symbols mean the same as above.

2.6 Analysis and Design

2.6.1 Loads and Load Effects

The analysis of a floating structure is very much needed and some special characteristics when compared to a land based structure (Clauss et al., 1992). There are various reasons for such characteristics to be included, namely:

a) The horizontal forces due to waves are several times larger than the horizontal forces on land based structures.

b) Horizontal loads on the floating structure depend on how the structure is moored to the seabed, whether it is by a rigid connection or a compliant connection.
    i. Rigid connection would prevent all horizontal movement.
    ii. Compliant connection would allow maximum horizontal movement of the structure of the order of the wave amplitude.

c) In a floating structure, the static vertical loads are carried by buoyancy. With the mooring lines, the horizontal forces are balanced by the inertia forces. Moreover, if the horizontal size of the structure is larger than the wave length, the resultant horizontal forces will be reduced due to wave forces on the different structural parts will have different phase (direction and size). This would then render the mooring system to have a small force relative to the wave forces. Therefore the mooring system would function to prevent a drift off of the structure.

d) Sizing of the floating structure and its mooring system depends on the function of that structure. Also, it is important to consider environmental conditions in terms of waves, currents, and wind.

e) It is important to consider accidental loads possible by ship impacts and ensure the overall safety is not threatened by a possible progressive failure due to such damage.

In summary, the loads that are important to be considered are:

a) Dead load
b) Hydrostatic pressure (including buoyancy)
c) Live load
d) Abnormal loads (impact loads from ships, etc)
e) Earth pressure on mooring system (dolphins, wind load, effects of waves)
f) Earthquake effects (including dynamic water pressure)
g) Temperature effects
h) Waves, current and wind loads
i) Effects of tidal change
j) Ship waves
k) Erecting loads

However, the main point of this study is the effects of the wave forces that will be acting on the floating breakwater.

2.6.2 Design Considerations for Floating Body

A floating structure design is very much different compare to a land-based structure. The design of a floating structure must meet the operation conditions, strength, and serviceability requirements, safety requirements, durability, visually pleasing to the environment, and cost-effective (Watanabe et al., 2004). The design consideration can be listed down to: materials, dynamic stability, design criteria, and corrosion protection.

2.6.2.1 Materials

Materials are very important to prevent high maintenance cost, i.e. cleaning up the corrosion. The materials suitable for corrosive sea environment should be high performance reinforced concrete, containing fly ash and silica fume for water-tightness of concrete to prevent corrosion of reinforcements. The steel used for a floating structure should satisfy the appropriate standard specifications (such as the Technological Standard and Commentary of Port and Harbor Facilities 1999).

2.6.2.2 Stability

Stability is the quality, or the attribute of an object to be stable. In other words, it is the quality of being free from change or variation. Stability conditions are important to the configuration of a floating structure. To determine the stability of the floating breakwater, the righting moment of the structure has to be determined. Usually the floating structure would be stable regardless of whether the centroid of the structure is above or below the centre of buoyancy (E. John Finnemoer et. el., 2002).

2.6.2.3 Design Criteria

A good design would to be ensured by designing the structures to comply with serviceability and safety requirements.

Serviceability criteria are introduced to ensure the structure fulfils the function required, and are specified by the owner. These criteria are related to motions and structural deformations. Therefore, it is noted that these criteria are including displacements, velocities, and accelerations.

Safety requirements are imposed to avoid any injuries, environmental impacts, property damages, and in worst case scenarios, fatalities. An important design issue regarding safety is the evacuation and rescue. Based on this, one good method for safety measures is to provide a safe place for people that were involved before a safe escape can be made.

One of the major failure modes for a floating structure is the drift-off when the mooring line fails. Therefore a proper risk analysis should be made to ensure all possibilities of the failure can be assessed. Another major failure mode would be the sinking of the floating structure. Therefore it is also important to assess based on that criteria. Other criteria would to be the overall stability due to overturning moments by wind, and uprighting moment due to hydrostatic of the inclined body.

Therefore modern safety criteria for marine structures are expressed by limit states as indicated in Table 4.1 (Moan, 2004).

Table 2.3 Safety criteria

Limit states	Description	Remarks
ULS	- overall 'rigid body' stability - ultimate strength of structure - ultimate strength of mooring system	Component design check
Fatigue (FLS)	- failure at joint - normally welded joints in hull and mooring system	Component design check depending on residual system strength after fatigue failure
Accidental (ALS)	ultimate capacity of damaged structure (due to fabrication defects or accidental loads) or operational error	System design check

2.6.2.4 Corrosion Protection

To protect the steel in the structure has many methods, including coatings, cathodic protection, corrosion allowance, and corrosion monitoring (Watanabe et al., 2004). However, overprotection should be avoided because it may cause hydrogen embrittlement. Also, in areas where marine growths are significant, antifouling coatings should be considered. These protections should be in accordance to specifications such as NACE Standard RP-01-76.

CHAPTER 3

METHODOLOGY

3.1 Experimental Setup

The experimental setup consists mainly of four equipments, namely the wave probes, pressure transducer, velocitymeter, and a video camera. The experimental setup will be done in the wave flume of Universiti Teknologi PETRONAS under controlled environment where tides and currents are not taken into account, very much like the conditions applied during the experimental studies of the WSS. Once again, the experiment was done in the 12m long, 0.3m wide, and 0.45m deep wave flume in the hydrology laboratory of Universiti Teknologi PETRONAS. The waters in the wave flume was subjected to monochromatic waves created by the wave maker of the flume ranging from 0.85seconds to 1.50seconds wave periods in 0.2m and 0.3m water depth. Generally, in this study, only the wave forces applied to the floating breakwater were studied.

3.1.1 Wave Flume

The wave flume of Universiti Teknologi PETRONAS is located at the hydrology laboratory and has a dimension of 12m x 0.3m x 0.45m, the length, width, and depth respectively. The wave flume is shown in Figure 3.1 below. The wave flume has a rigid steel bed and has glass panels which are properly sealed lined up at the entire length of the flume. The glass panels are for the purpose of observation of experiments.

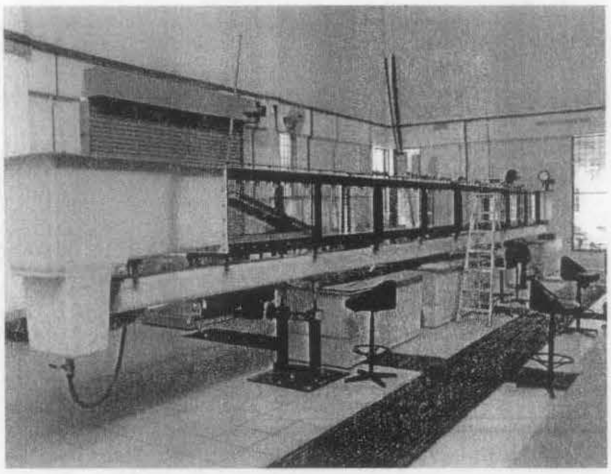


Figure 3.1 Wave flume in the hydrology lab of Universiti Teknologi PETRONAS

3.1.2 Wave Maker

The wave maker is placed at the far end of the flume as shown in Figure 3.1. The wave maker is shown in Figure 3.2. Also shown in Figure 3.2 is the switch box which controls the pump that pumps water around the flume, the frequency knob that controls the wave maker, and the power knobs. As shown in Figure 3.2, the frequency knob at the switch box would allow adjustments to the rotational speed and thus the stroke frequency of the wave maker and thus controlling the flat movable paddle to create monochromatic waves.

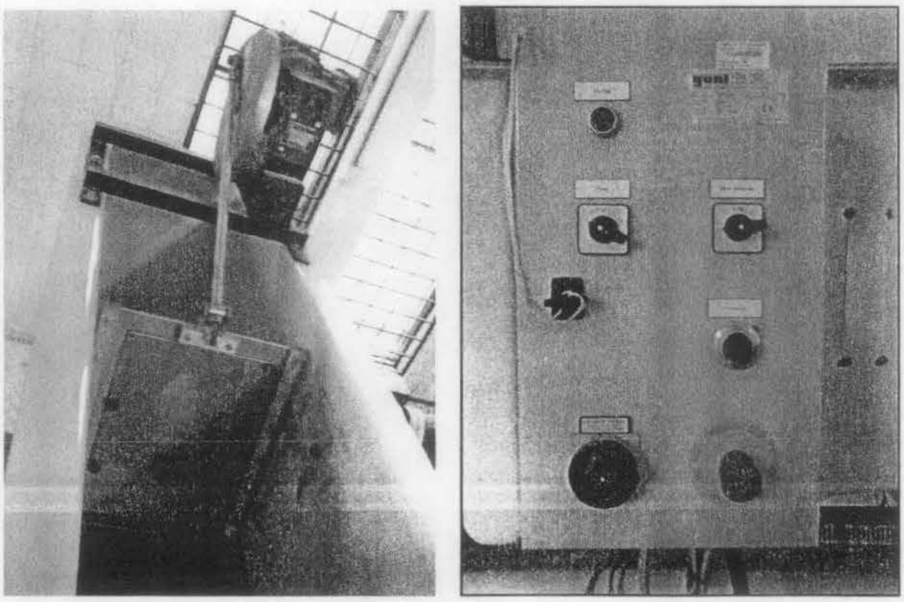


Figure 3.2 (a) Wave maker of the wave flume, (b) Switch box

3.1.3 Wave Absorber

Part of the flume which is very important is the wave absorber. The wave absorber is located at the near end of the flume as shown in Figure 3.1 and it is at the opposite end of the wave maker. The wave absorber is located at such place, which is the reflective boundary of the wave flume to attenuate incoming wave energy through various wave dissipation mechanisms. The material of the wave absorber is wire mesh and is mounted on a steel plate that is adjustable from a slope of 0° to 90°. Throughout this study, the wave absorber is placed at a 15° slope. Figure 3.3 shows the placement of the wave absorber.

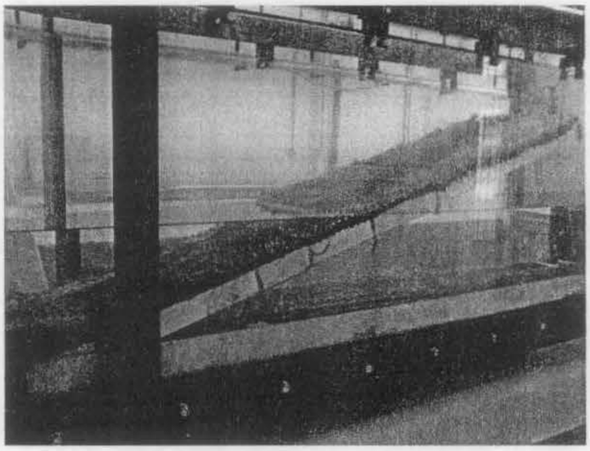


Figure 3.3 Wave absorber of the wave flume

3.1.4 Water Pump

The water pump is placed near the wave maker and it is at the bottom of the flume. The water pump plays a major role as it is responsible to pump water around the wave flume. The water pump is controlled using the valve located at the pump with the power switch at the switch box shown in Figure 3.2. Figure 3.4 shows the placement of the water pump.

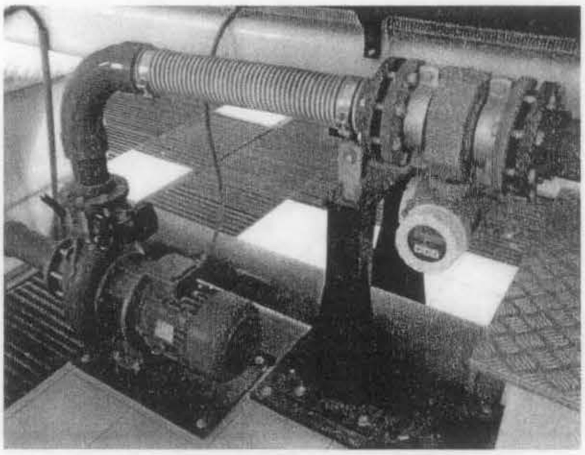


Figure 3.4 Water pump of the wave flume

3.1.5 Wave Probes

There are many physical and application variables that affect the selection of the optimal level monitoring solution for industrial and / or commercial processes. One of these types of level measuring is by using wave probes. Currently in the hydrology lab of Universiti Teknologi PETRONAS, the instrument used is a conductive based level sensor. The technology behind conductive level sensing involves a low-voltage, current-limited power source applied across separate electrodes. The power supply is matched to the conductivity of the water. The power source frequently incorporates some aspect of control, such as high-low or alternating pump control. A conductive liquid (i.e. water) contacting both the longest probe (common) and a shorter probe (return) completes a conductive circuit. Conductive probes have the additional benefit of being solid-state devices and are very simple to install and use. Figure 3.5 shows the wave probe.

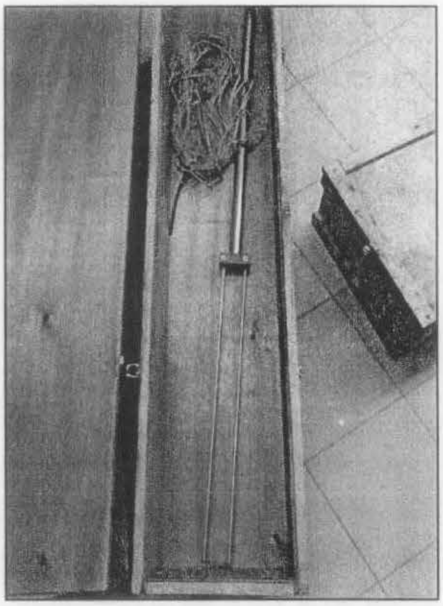


Figure 3.5 Wave probe

3.1.6 Pressure Transducer

A transducer is a device that converts one type of energy to another for various purposes, including measurement or information transfer. In a broader sense, a transducer sometimes defined as a device that converts a signal from one form to another.

When water flows are not in equilibrium, local pressures may be higher or lower than the average pressure in a medium. These disturbances propagate from their source as longitudinal pressure variations along the path of propagation. This is also called sound. Sound pressure is the instantaneous local pressure deviation from the average pressure caused by a sound wave. Sound pressure can be measured using a hydrophone in water. The effective sound pressure is the root mean square of the instantaneous sound pressure over a given interval of time. From this sound pressure, an electrical signal will be send to a display unit, enabling to read the pressure. Figure 3.6 is the pressure transducer that would be used in the experiments.

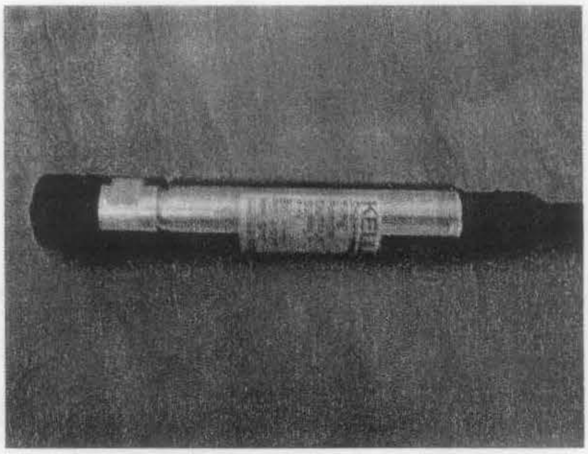


Figure 3.6 Pressure Transducer

3.1.7 Velocitymeter

The velocitymeter is a gauge to measure flow of a liquid. Flow measurement is the quantification of bulk fluid movement. The probe that is used in hydro lab uses the principle of ultrasonic flow meter.

The probe is placed statically in the path of the flowing water. As the water passes the probe, disturbances in the flow called vortices are created. Inside the probe, there is a piezoelectric crystal, which produces a small but measurable voltage pulse every time a vortex is created. The frequency of this pulse is also proportional to the water flow rate, and it is measured. This frequency will be sent to the display unit and converted into a simple velocity graph in three directions, x-axis, y-axis, and z-axis for the interpretation of the user. Figure 3.7 is an example of a velocitymeter.

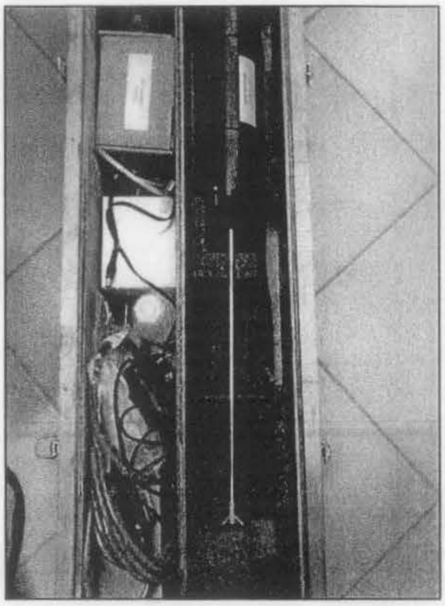


Figure 3.7 Velocitymeter

3.1.8 Video camera

The video camera will be used as a basic video camera function to record the behavior of the floating breakwater. The video camera used as shown below in Figure 3.8.

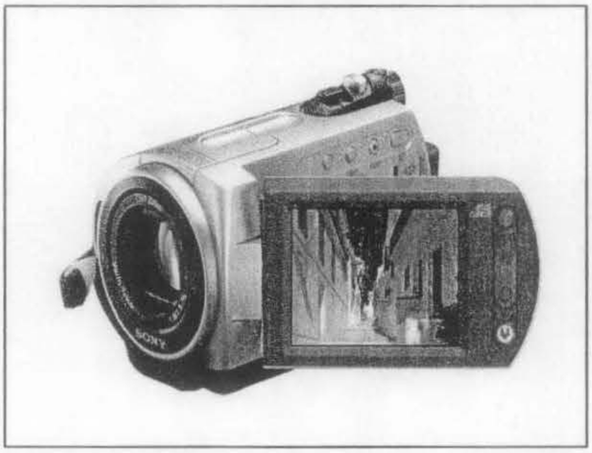


Figure 3.8 Video camera

These equipments are mainly used to measure the wave parameters acting on the floating breakwater by the means of transmitting a voltage signal. Training was conducted last semester in the hydrology lab of Universiti Teknologi PETRONAS for familiarity of the equipment. The main equipment that will be used would be the pressure transducer to measure the pressure acting on the floating breakwater. The wave probes as well as the velocitymeter will be less used as this study is mainly focusing on the forces acting on the floating breakwater, not the wave attenuating properties. The video camera will be used to record the behavior of the floating breakwater during the experiments for observation. These observations include the dynamic properties of the floating breakwater based on the theory mentions above.

3.2 Experimental Plans

This study would include a minimum of sixteen different variations for experiments. The experiments are set up to evaluate the wave forces acting on the previously designed floating breakwaters. These tests include the study of effects of geometry towards the aptitude against wave forces, various wave climates towards the floating breakwater, a variety of fastening system, and various water depths.

Currently there are two different geometries for a floating breakwater design in Universiti Teknologi PETRONAS, namely the GEN-1 and GEN-2 design of the "Wave Suppress System". The effectiveness of these designs was previously tested. These geometries will be re-tested again, the difference being made that the study will be done to evaluate the wave forces acting on the breakwaters that were previously designed.

The wave climates will include the difference of wave heights, wave period, and thus, the wave energy acting towards the floating breakwater. These tests are important for the design of the floating breakwater's strength, whether it is material or geometry sense. Therefore, the wave climates will be tested in the usual climate, as well as the extreme climate. The extreme wave climate will be tested to ensure the capacity of the design towards unpredictable wave conditions.

The fastening system that can be tested would be the pile, also known as the fixed mooring system, as well as the cable (tether), also known as the flexible mooring system. Both of these will be tested to known the effectiveness of both systems, rendering the designer to know which mooring system is more effective.

Lastly, the experiments will be done in various water depths. The summary of the experiments is listed down in Table 3.1.

Table 3.1 Experimental Plans

Fastening System	Geometry	Wave Height	Water Depth
Experiment 1: Fixed (Pile)	Model 1	Normal wave height	20 cm
			30 cm
		Extreme wave height	20 cm
			30 cm
	Model 2	Normal wave height	20 cm
			30 cm
		Extreme wave height	20 cm
			30 cm
Experiment 2: Flexible (Chain / Cable)	Model 1	Normal wave height	20 cm
			30 cm
		Extreme wave height	20 cm
			30 cm
	Model 2	Normal wave height	20 cm
			30 cm
		Extreme wave height	20 cm
			30 cm

The methodology of the experiment can be summarized as below:

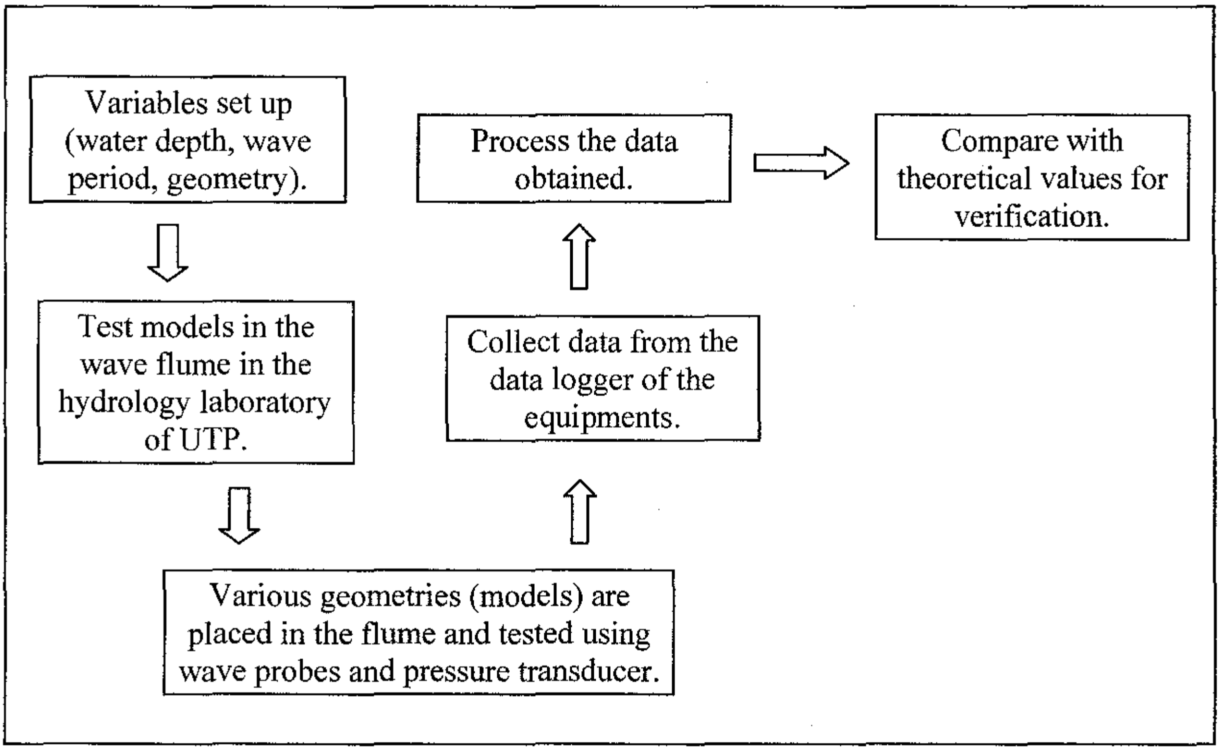


Figure 3.9 Simplified Methodology of the Study

However, unforeseen problems occurred and the methodology described above has been rendered useless. The main problem is that the equipments mentioned above could not be used in this experimental study as the equipments were not able to connect with the host computer; therefore the data from the equipments were not made unavailable to the author. Therefore, another methodology was proposed and implemented. Henceforth, the methodology mentioned above is known as *Methodology I*, and the methodology that will be mentioned in the following section is known as *Methodology II.*

3.3 Methodology II

Because of the faulty connection that occurred between the equipments and the host computer, *Methodology I* was not able to be implemented. Therefore *Methodology II* was improvised based on material science.

Initially the piles that were supposed to be used were steel piles that were rigid and tough. Hence this would restrict movement of the piles and therefore the floating breakwater. Using the steel piles, the movement of the floating breakwater would be restricted from six degrees of motions to only one degree of motion, which is the vertical motion, rendering the floating breakwater to be only able to move up and down. However, this would to be used if the equipments were in commission. Therefore another alternative material of pile which is more flexible is selected.

The alternative pile that was selected was made of cane (Figure 3.10). This pile is flexible and the movements of the floating breakwater were restricted from six degrees of freedom to two degrees of freedom. These movements are about the translational axes, namely the ability to move upwards/downwards, and forward/backward. The material is selected due to its flexibility, and a separate experiment based on material science can be done. The cane pile is subjected to loadings (weight) and the displacement of the pile due to these loadings are observed and recorded. With these data, a Force vs. Displacement graph was obtained. Then, the experiment continues with the usual experiment, using the cane pile instead of the steel pile in the wave flume. The pile is then subjected to incoming waves, and the forward/backwards displacement is recorded. This data is then compared with the Force vs. Displacement graph to obtain the force that is subjected to the floating breakwater. However, because of the limited choices and time due to the drawback by the failure of equipments as mentioned above, only cane piles were subjected to the study and there were no cables involved.

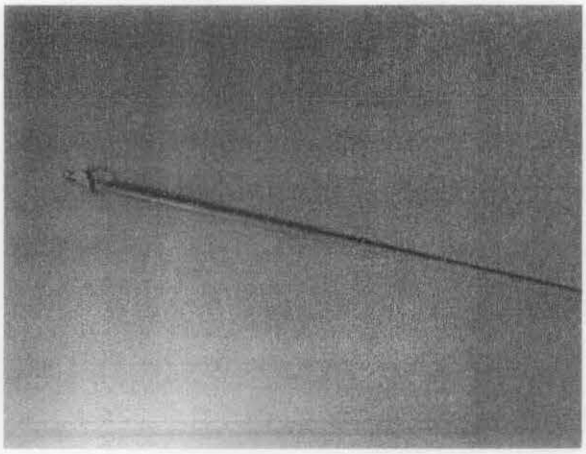


Figure 3.10 The cane pile used in this study

Methodology II of the experiment can be summarized as below:

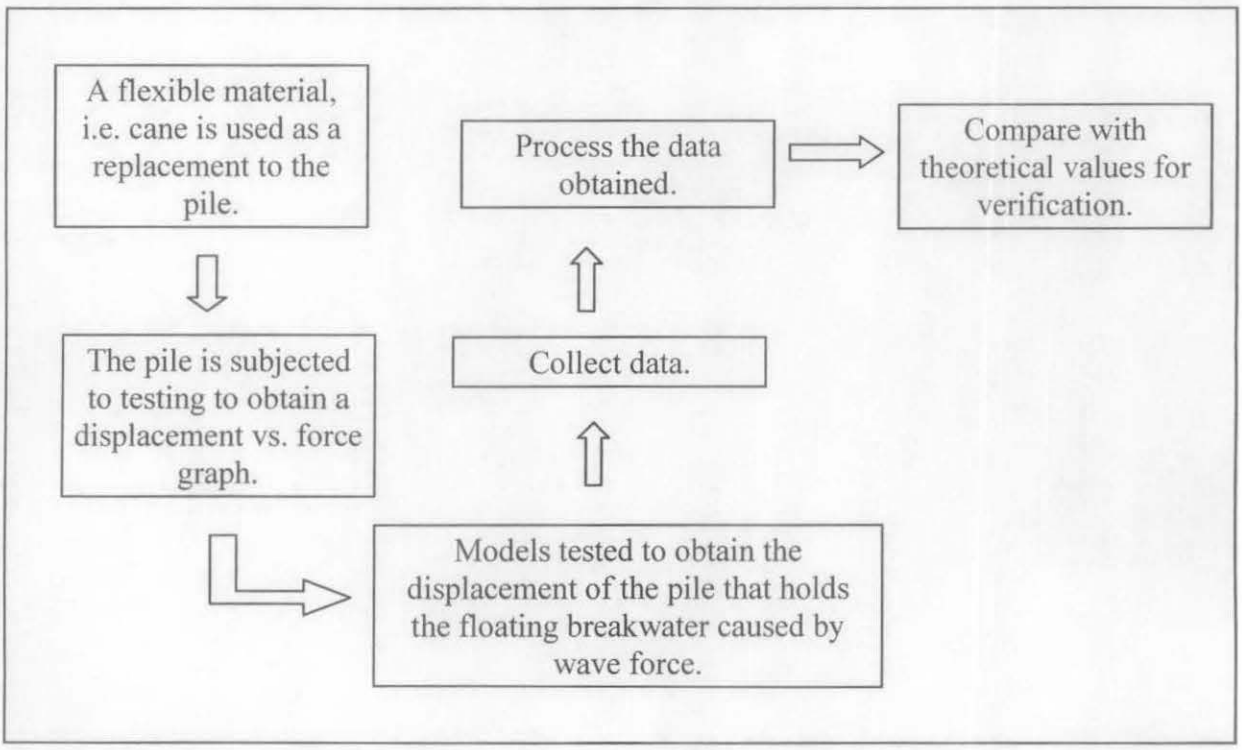


Figure 3.11 Simplified diagram of *Methodology II* of the Study

The theoretical pressure values mentioned in both *Methodology I* and *Methodology II* are obtained from Goda's Principle for Breakwater Design (1992) as mentioned in Chapter 2 in Section 2.5.

3.4 Safety Hazards

Safety hazards are part of the requirements that is needed to be well understood in managing the final year project. The definition of hazard is any source of potential damage, harm, or adverse health effects on something or someone under certain conditions of work (CCOHS, 2005). A hazard is basically something that can cause harm both direct and indirectly. In the sense of this project, the safety hazards can be broken down into two major points, namely the safety hazards in the hydrology laboratory during the experimental phase of the project, and computer ergonomics in the analysis phase of the project.

3.4.1 Safety Hazards in the Hydrology Laboratory

Accidents in the laboratory usually occur due to carelessness or ignorance by the user. The laboratory hazards can be grouped into physical hazards and physical agents.

The physical hazards are those hazards that involve the physical body being exposed to danger due to human factors such as not observant, careless, etc. These hazards include:

a) Slips and trips

The hydrology laboratory involves water during the experiments. Sometimes spills may happen causing the floor to be slippery. Also, there are objects around the area of the lab that could cause a person to trip if not careful.

b) Falls from height

The flume is at a height where a normal person would not be able to reach, therefore ladders and chairs are used to reach the top of the flume to make necessary changes to the configuration of the model in the flume (draft, mooring system, etc.)

c) Dangerous machinery

The flume itself is a machine that has a pump for pumping the water around the flume, and a wave maker that oscillates in the flume. Injury may occur if the user is not careful at the moving parts of the flume.

d) Electricity

The flume itself is a machine; therefore electricity is needed to operate it. Therefore electrical shock may occur because water and machinery makes the laboratory even more hazardous.

Physical agents are sources of energy that may cause injury or disease. These include noises, lighting and vibration in the hydrology laboratory.

a) Noises

A common usage of the term noise is the unwanted sound that would lead to irritation, and psychosocial stress. Noise can block, distort, or change a meaning of a message in human communication. The machinery in the laboratory would create noise and therefore in long term, cause psychosocial stress in the users.

b) Lighting

Lighting is the light sources that come from lamps, or natural illumination of interiors from daylight. Without proper lighting, the task performance would be lowered. Not only that, without proper lighting, health effects such as the sight would be adversely affected. Over illumination can lead to adverse health and psychological effects such as headache, stress and increased blood level as well. Also, glare would decrease work efficiency.

c) Vibration

Vibration is an oscillation about an equilibrium point. In the case of this experiment, the vibration is periodic. The vibration is created by the oscillation of the wave maker, creating unnecessary sound – noise. Also, the vibration can create pressure waves. In turn, the pressure waves can generate vibration of structure (e.g. ear drums). With this, psychosocial stress can be induced by vibration.

With the physical hazards and the physical agents describe above, the user in the laboratory during experiments should pay extra attention to the hazards that are present. This would involve placing the objects and items in proper place, be careful while using the equipments, and wear proper clothing while performing the experiment.

In short, the preventive measures are stated below:

a) Eliminate the hazard

   This is the most effective measure. The best way is to choose a different process or modify the current job process.

b) Contain the hazard

   If the hazard cannot be eliminated, it should be contained.

c) Revise work procedures

   Steps that are hazardous should be considered to be modified for safety purposes.

d) Reduce the exposure

   This is the least favorable method. This method is only used when there are no other solutions.

3.4.2 Computer Ergonomics

Ergonomics is the branch of engineering science in which biological science is used to study the relation between workers and their environments. In other words, ergonomics are applied to maximize productivity by reducing operation fatigue or discomfort. A major contribution to poor performance is due to poor or static posture.

During the analysis phase of this project, the works are mainly done at desk, meaning the works are being done using computers, or writing and taking down notes and pointers in a notepad. Without proper ergonomics, repetitive strain injury might occur. Sitting in the same posture for a long period of time, with arms flexed, typing without proper breaks and a poor posture increases the static loading on a number of muscles throughout the body. Over time these muscles tend to become shorter and denser, impeding blood circulation to the arms, neck and shoulders.

Therefore, it is important to be aware of the ergonomics while working with a computer. Firstly, the head and shoulders should be relaxed while working. Also, the monitor of the computer should be at least an arm's length away and 15° to 30° below the line of sight. Lighting is very important, and it should not have direct shine to the monitor of the eyes. The forearms and the thighs should be 90° from the spine, and the user should sit on a comfortable chair. The rest of proper ergonomic is shown in Figure 3.5 below:

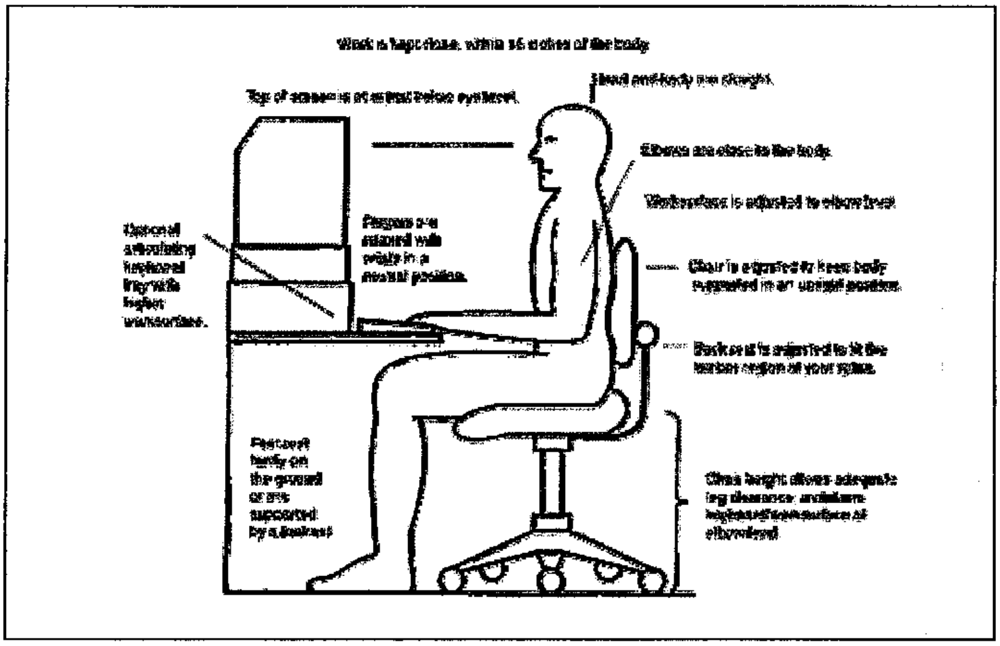


Figure 3.12 Proper computer ergonomics (http://ergo.human.cornell.edu/)

CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 Summary of Tests

The tests done for this study with the corresponding values are shown in *Appendix A*.

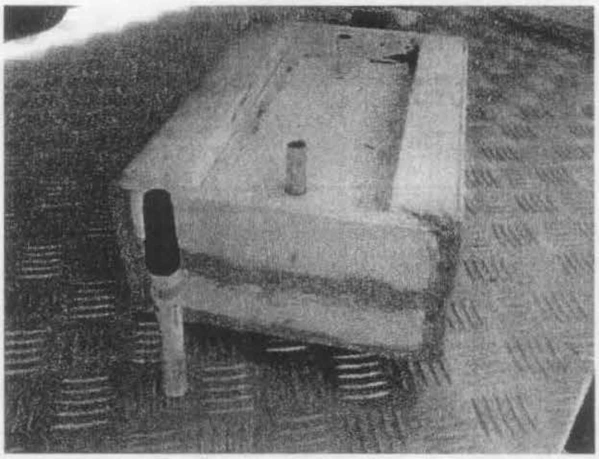
4.2 Description of the Model

Unlike the models in previous Final Year Projects that used solid lightweight concrete as the material of the models, this study involves models that are made of wood, and they are hollow. The justification of the models being hollow is to toy around with the weight, and therefore the draft of the models to determine the stability and the best configuration of the models. Also, the hollow sections allow the user of the models to toy about with the weight distribution to determine the stability of the models. The joints of the model were sealed with silicon sealant, and painted with a coating of water proof painting. Instead of having one piling point in the middle of the models like previous studies, this study changed the piling system to have two piling points. This is to enhance the stability of the models.

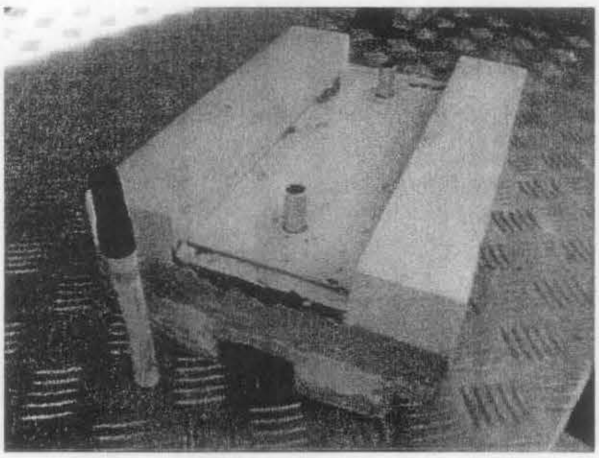
Because the models are made of wood, they are very light. Because of that, the models cannot achieve the desired draft to compare with the previous models' results. The calculations to determine the extra weight needed for the desired draft is shown in the *Appendix B*. The weight of the GEN-1 model is 0.92kg, having needing another 2.68kg for a desired draft of 6cm. Therefore, the total weight of the GEN-1 model is 3.6kg. The weight of the GEN-2 model is 1.1kg, needing another 2.05kg to obtain a desired draft of 6.5cm. The total weight of GEN-2 model is therefore 3.15kg. GEN-3 has a weight of 0.91kg and need another 0.78kg to obtain a desired draft of 4.9cm. the total weight of GEN-3 model is 1.69kg.

The configuration of the models is based on previous studies, namely GEN-1, GEN-2, GEN-2 with keel, and a control unit taking the shape of a box. GEN-1 model has two arms of 5.00cm, and two bottom legs which have 90° side slope with the width of 5.00cm as well. GEN-2 on the other hand, has two arms of 4.00cm, the two legs has been enhanced into 45° slanting side slopes. The box unit has a dimension of 0.2m x 0.1m x 0.3m. From here forth, the control unit will be known as M-1, GEN-1 is known as M-2, GEN-2 is known as M-3, and GEN-2 with keel is known as M-4.

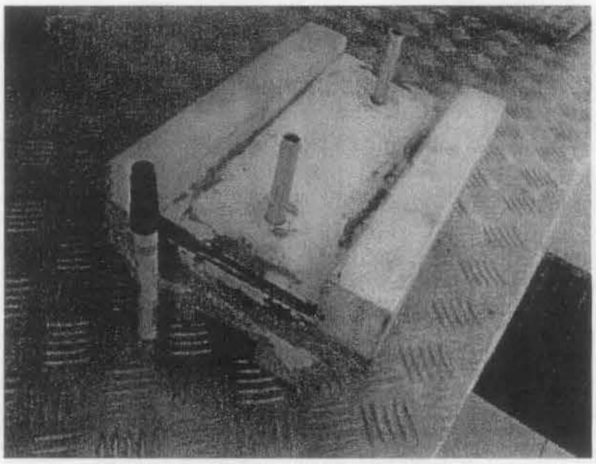
Figure 4.1 shows this study's models.



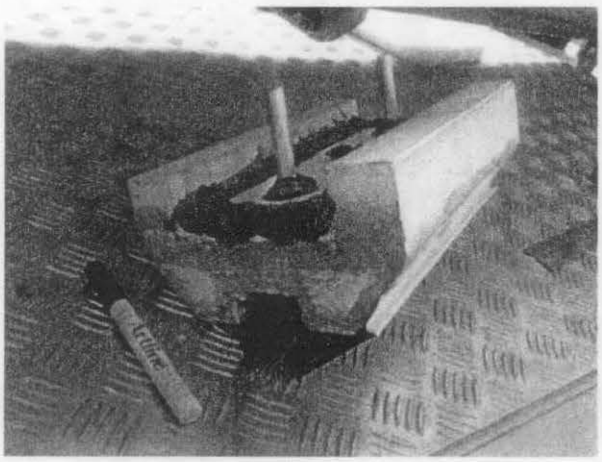
(a) Model M-1



(b) Model M-2

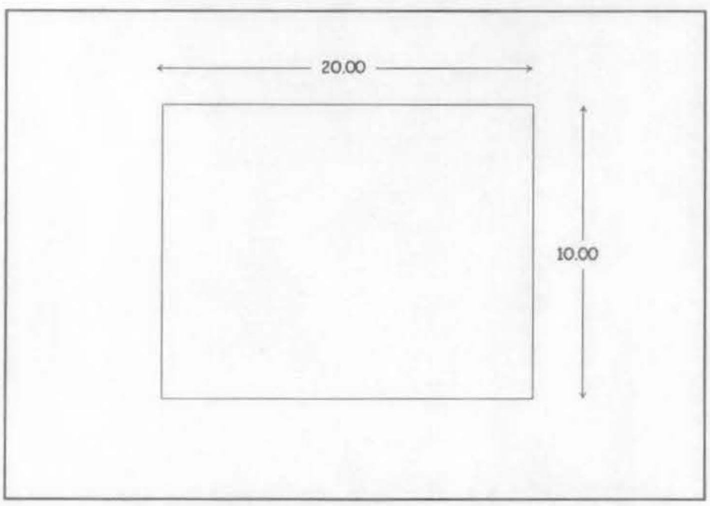


(c) Model M-3

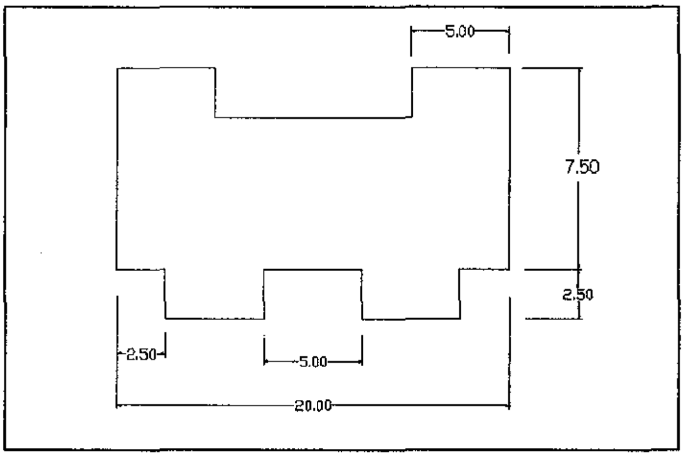


(d) Model M-4

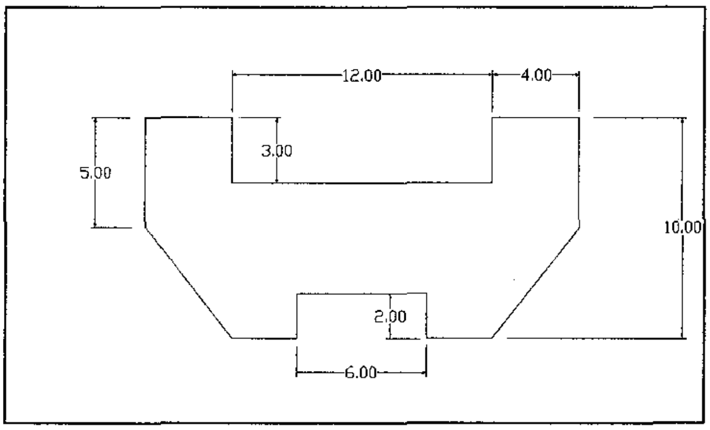
Figure 4.1 Models that will be used in the study



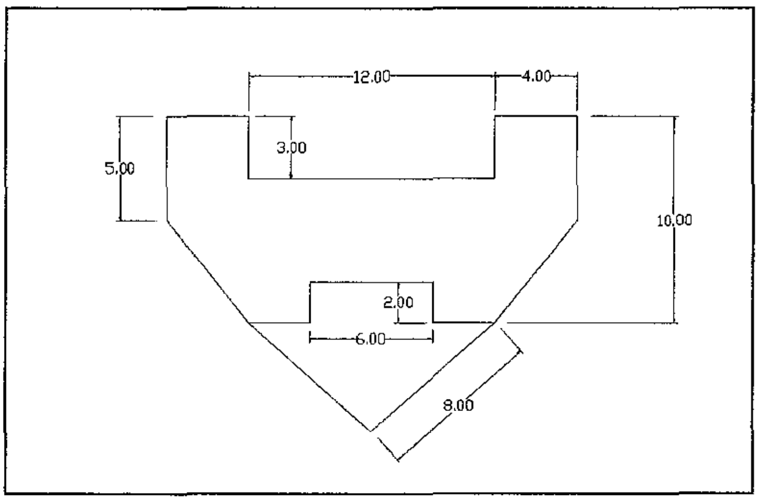
(a) Model M-1



(b) Model M-2



(c) Model M-3



(d) Model M-4

*All units are in cm

*All models has a length of 30cm

Figure 4.2 Schematic drawing of the models that will be used in the study

4.3 Stability

Stability is the quality, or the attribute of an object to be stable. In other words, it is the quality of being free from change or variation. Stability conditions are important to the configuration of a floating structure. To determine the stability of the floating breakwater, the righting moment of the structure has to be determined. Usually the floating structure would be stable regardless of whether the centroid of the structure is above or below the centre of buoyancy (Finnemoer et. el., 2002).

4.3.1 Determination of Stability

There are different strategies to solve for the stability problems. One of the strategies is to employ the use of the metacentric height to determine the stability of the floating body. Metacentric is to have two equal arms because of the median position of the body. The method employed is discussed as below (Saiedi, 2007):

a) From the principles of Archimedes, the geometry of body and density of fluid and body are to be equate; in other words, the weight of displace fluid is suppose to be equal to the total weight of the body. With this principle, the depth of immersion of the body, better known as the draft of the floating body, or the weight of the body can be found by having known one of the unknown.

b) After determining the draft of the floating body, the stability could be determined. To assess this stability, the centroid of the body should be first found.

c) Then, the centre of buoyancy should be found.

d) Find the distance between the centroid and the centre of buoyancy, GB.

e) The righting moment from the centre of buoyancy, MB, can be found using the equation $MB = I/V_s$, where I is the second moment of inertia and V_s is the volume of the submerge part of the body or the volume of the fluid displaced.

f) With the values of GB and MB, calculate the metacentric height, using the equation $MG = MB - GB$.

With the value of *MG*, we can determine the stability of the floating body. If the metacentre, M, lies above the centroid, then the body is stable. This would yield a positive value of MG. In other words, the metacentric height yields a positive number. On the other hand, if the metacentric height yields a negative number, the floating body is therefore unstable.

The calculations of the stability of all models are presented in *Appendix B*. The stability of all the models is tabulated as below:

Table 4.1 Stability of the models

Model	Stability	
	Stable	**Instable**
M-1	/	
M-2	/	
M-3	/	
M-4	/	

All the models are stable based on theoretical calculations. The calculations were then verified by the experiments done in the wave flume in the hydrology laboratory.

4.4 Determination of Wave Period, *T*

The determination of wave period, T is with respect to different stroke frequencies. Based on definition, wave period is defined as the time required for a crest to travel a distance of one wave length (Chakrabarti, 1987). In the hydrology laboratory, the average time taken for one complete cycle of the crank is recorded for a set of stroke adjustment for calibration purpose. The observed wave period is tabulated is in Table 4.2. Also, the relationship between stroke frequency and the observed wave period has been graphically illustrated in Figure 4.2. From the figure, it can be observed that the average wave period depends on the stroke frequencies instead of the stroke adjustments. It is also observed that the wave period decrease as the stroke frequency increases. The relationship can be written as such: $T = 95.658 f^{-1.1384}$

Table 4.2 Observed wave period

Stroke Frequency (rpm)	Observed Wave Period, T (s)			Average Wave Period, T (s)
	80mm stroke adjustment	140mm stroke adjustment	200mm stroke adjustment	
108	0.49	0.50	0.49	0.49
88	0.60	0.60	0.60	0.60
74	0.71	0.71	0.71	0.71
64	0.83	0.84	0.83	0.83
56	1.01	1.00	1.00	1.00
50	1.02	1.02	1.03	1.02
44	1.25	1.25	1.25	1.25
40	1.27	1.27	1.27	1.27
37	1.66	1.65	1.66	1.66
34	1.68	1.68	1.68	1.68
31	1.70	1.70	1.70	1.70
29	2.45	2.42	2.42	2.43
27	2.47	2.45	2.46	2.46
25	2.51	2.50	2.50	2.50
24	2.49	2.50	2.50	2.50

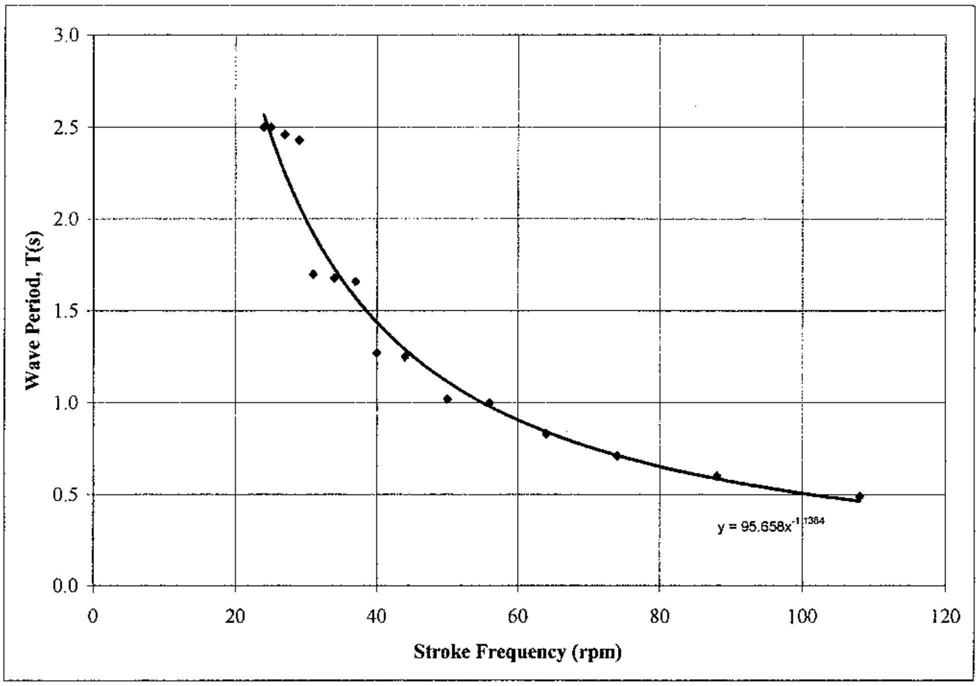


Figure 4.3 Average observed wave period for 80mm, 140mm and 200 strokes adjustment

Table 4.3 Wave period used in the study obtained from the relationship as above

Stroke Frequency (rpm)	Wave Period, T(s)
63.38	0.85
59.69	0.91
57.48	0.95
50.53	1.10
48.97	1.14
48.60	1.15
46.81	1.20
45.16	1.25
42.21	1.35
40.88	1.40
38.48	1.50

4.5 Determination of Water Condition

The determination of the water condition is important to know the performance of the structure acting at the depths. Water conditions can be divided into shallow water, transitional water and deep water. The following sections show the determination of the water condition.

4.5.1 Theoretical determination of Wavelength, L

The wavelength, L is determined with the reference from Table C-1 Shore Protection Manual of the US Army Corps of Engineer. The steps to obtain the wavelength are as below:

a) Determine the deepwater wavelength, $L_0 = \dfrac{gT^2}{2\pi}$

b) Determine d/L_0 for each water depth

c) Refer to Table C-1 of the Shore Protection Manual to obtain d/L

d) Classify the wave to deepwater, transitional water or shallow, based on the conditions below:

e) $d/L \geq 0.5 \rightarrow$ deepwater

f) $0.04 \leq d/L \geq 0.5 \rightarrow$ transitional water

g) $d/L \leq 0.04 \rightarrow$ shallow water

h) Determine the wavelength, L

Based on the steps above, the calculation of wavelength (shown in *Appendix B*) and the water condition with water depth of 20cm and 30cm is done and is tabulated in Table 4.3 and Table 4.4.

Table 4.4 Determination of wavelength for water depth of 20cm theoretically

Wave Period, T (s)	**d/L_0**	**d/L**	**Wavelength, L (m)**
0.85	0.1281	0.1650	1.2120
0.91	0.1077	0.1476	1.3548
0.95	0.0890	0.1313	1.5232
1.10	0.0786	0.1220	1.6399
1.14	0.0658	0.1010	1.8187
1.15	0.0598	0.1041	1.9204
1.20	0.0514	0.0956	2.0916
1.25	0.0461	0.0901	2.2209
1.35	0.0388	0.0819	2.4426
1.50	0.0320	0.0739	2.7066

Table 4.5 Determination of wavelength for water depth of 30cm theoretically

Wave Period, T (s)	**d/L_0**	**d/L**	**Wavelength, L (m)**
0.85	0.1922	0.2186	1.3725
0.91	0.1615	0.1907	1.5732
0.95	0.1335	0.1695	1.7699
1.10	0.1179	0.1528	1.9632
1.14	0.0987	0.1393	2.1536
1.15	0.0897	0.1281	2.3416
1.20	0.0771	0.1187	2.5275
1.25	0.0692	0.1106	2.7118
1.35	0.0581	0.1036	2.8947
1.50	0.0480	0.0921	3.2571

4.5.2 Determination of Wavelength, L by observation

Another method of observation was conducted to verify the accuracy of the calculated wavelength. The observed wavelengths are tabulated below in Table 4.5 and Table 4.6.

Table 4.6 Determination of wavelength (d = 20cm) based on observation

Wave period, T(s)	Wavelength, L(m)	d/L
0.85	1.0465	0.1911
0.91	1.1133	0.1796
0.95	1.0370	0.1929
1.10	1.2486	0.1602
1.14	1.0197	0.1961
1.15	1.4753	0.1356
1.20	1.4292	0.1399
1.25	1.6317	0.1226
1.35	1.7027	0.1175
1.40	1.6244	0.1231
1.50	1.4622	0.1368

Table 4.7 Determination of wavelength (d= 30cm) based on observation

Wave period, T(s)	Wavelength, L(m)	d/L
0.85	1.0395	0.2886
0.91	1.2894	0.2327
0.95	1.3374	0.2243
1.10	1.4201	0.2113
1.14	1.4925	0.2010
1.15	1.4530	0.2065
1.20	1.6076	0.1866
1.25	1.4519	0.2066
1.35	1.2571	0.2386
1.40	1.7865	0.1679
1.50	2.1254	0.1411

4.5.3 Water Condition

The water condition is determined based on the values of d/L. The water conditions are based on these criteria:

- d < L/25 Shallow water conditions
- L/25 < d < L/2 Transitional water conditions
- d > L/2 Deep water conditions

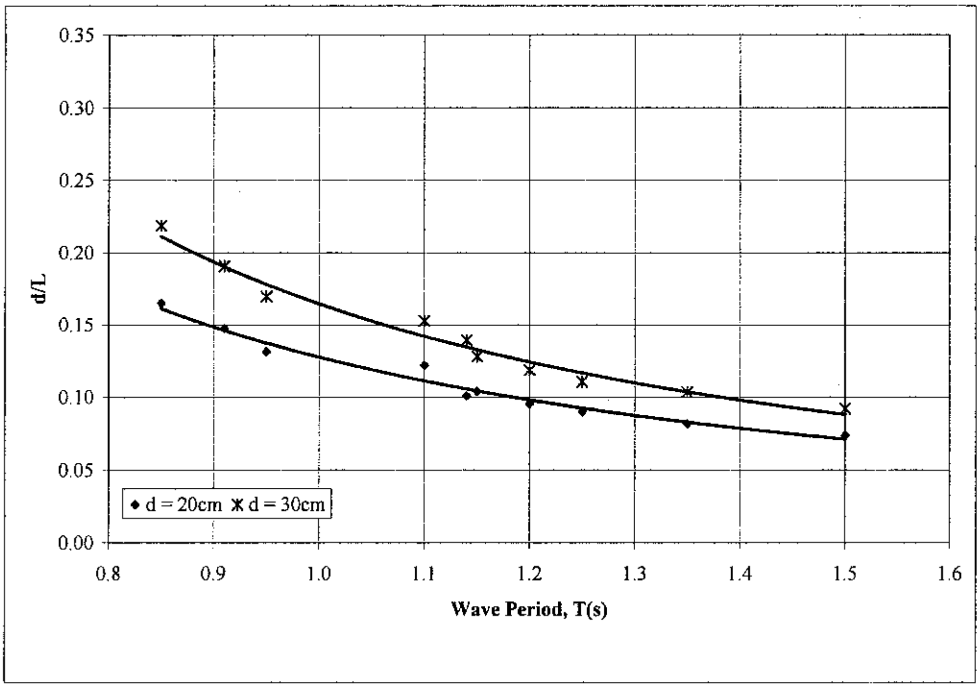


Figure 4.4 Theoretical classifications of water conditions

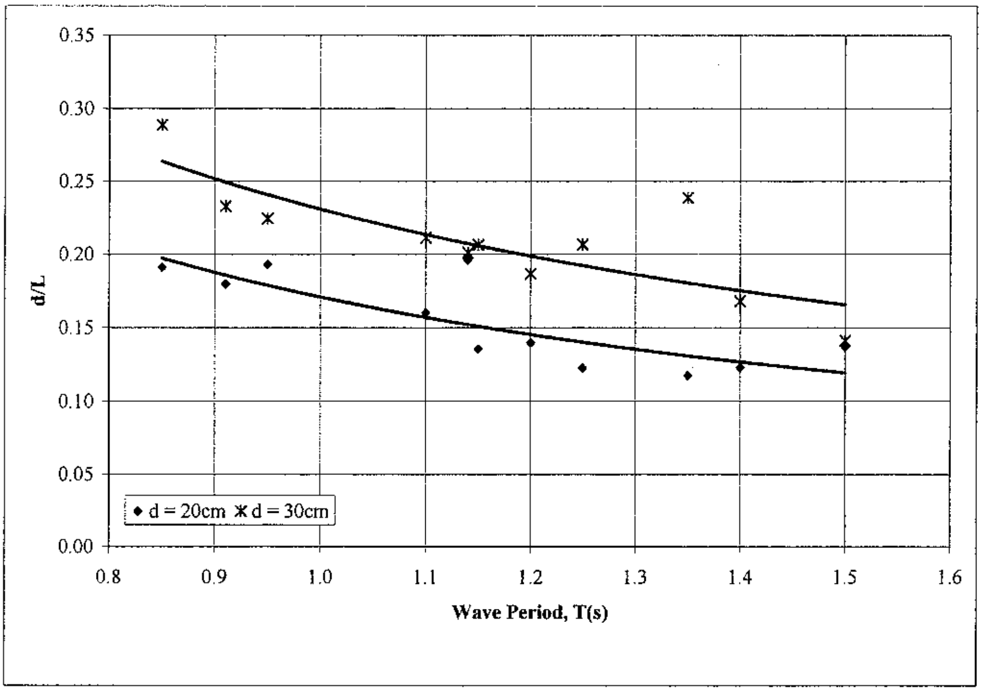


Figure 4.5 Classifications of water conditions by observations

Based on the figures above, the graphs showed that the water conditions based on the depths of 20cm and 30cm, with the wave period in between 0.85s and 1.5s are all within the range of 0.04 and 0.5. Therefore it can be concluded that the wave length by theoretical calculations and by observations are the same and that these conditions are within the transitional water condition boundaries. The error margin between the theoretical and observed waves was an average of 13% which is acceptable.

4.6 Determination of Incident Wave Height

An incident wave height is the wave height that approaches the floating breakwater. The incident wave height was determined by measuring the wave heights with the structure placed in the flume, and a scale was placed on the glass panel of the flume for observations. The incident wave heights for both 20cm and 30cm water depth for all models are graphically illustrated in Figure 4.5, Figure 4.5, Figure 4.6, and Figure 4.7.

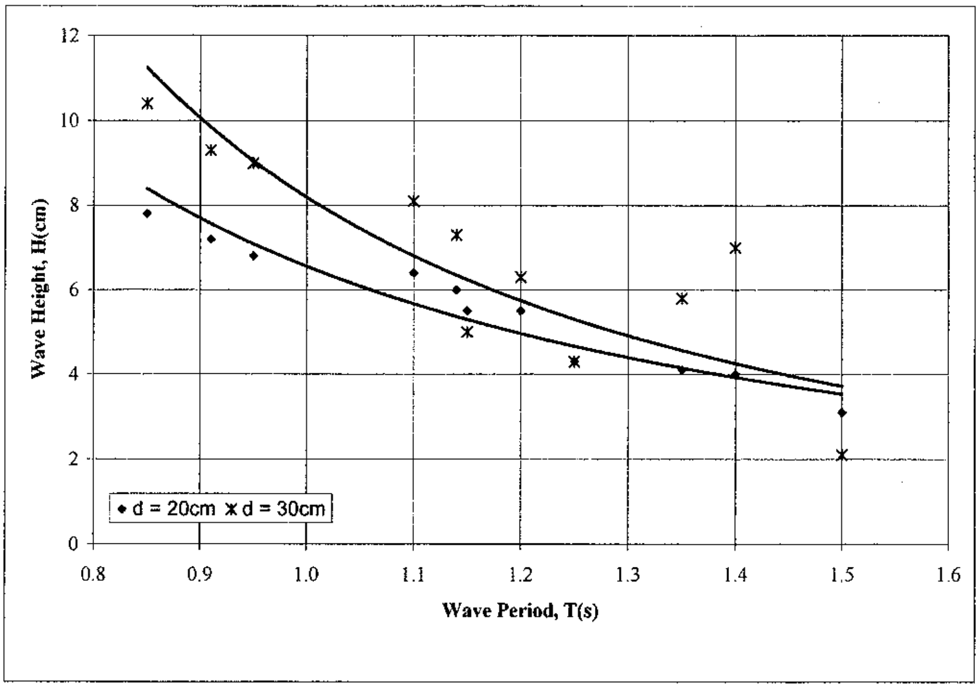


Figure 4.6 Wave Heights for M-1

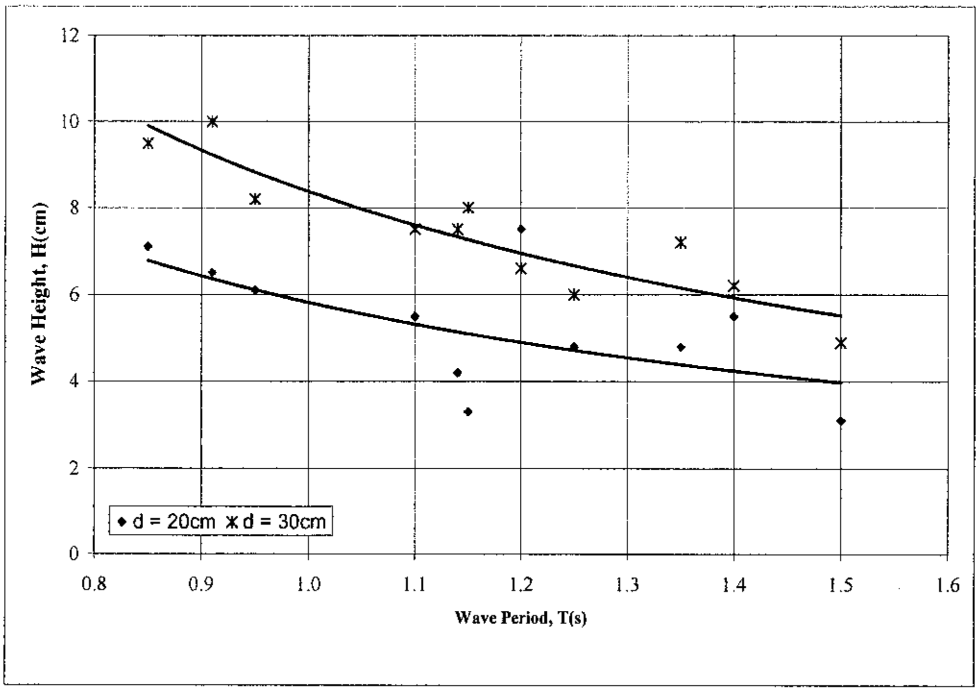


Figure 4.7 Wave Heights for M-2

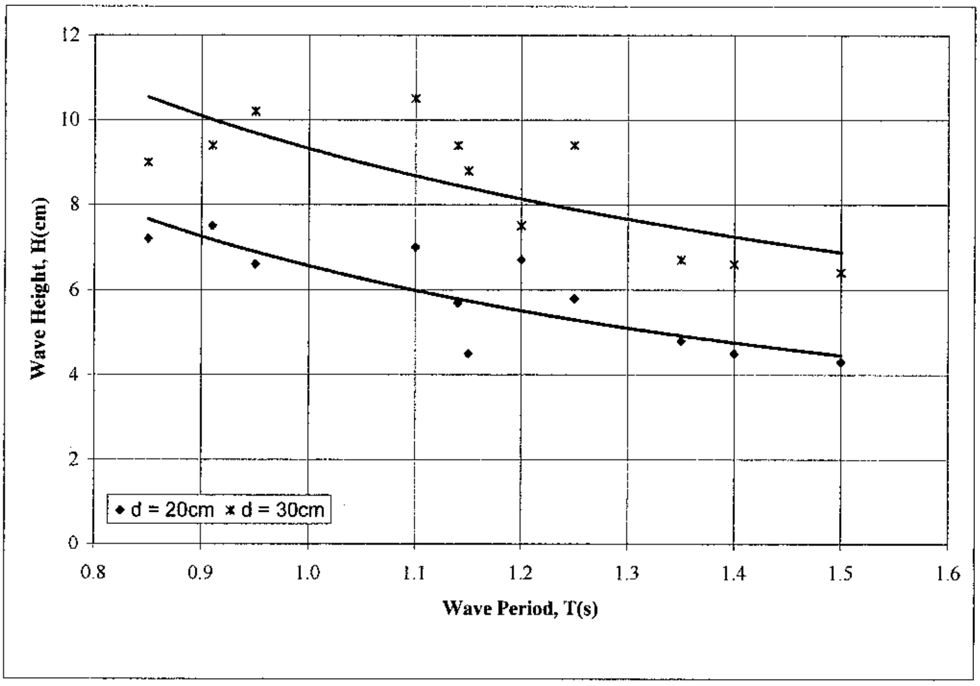


Figure 4.8 Wave Heights for M-3

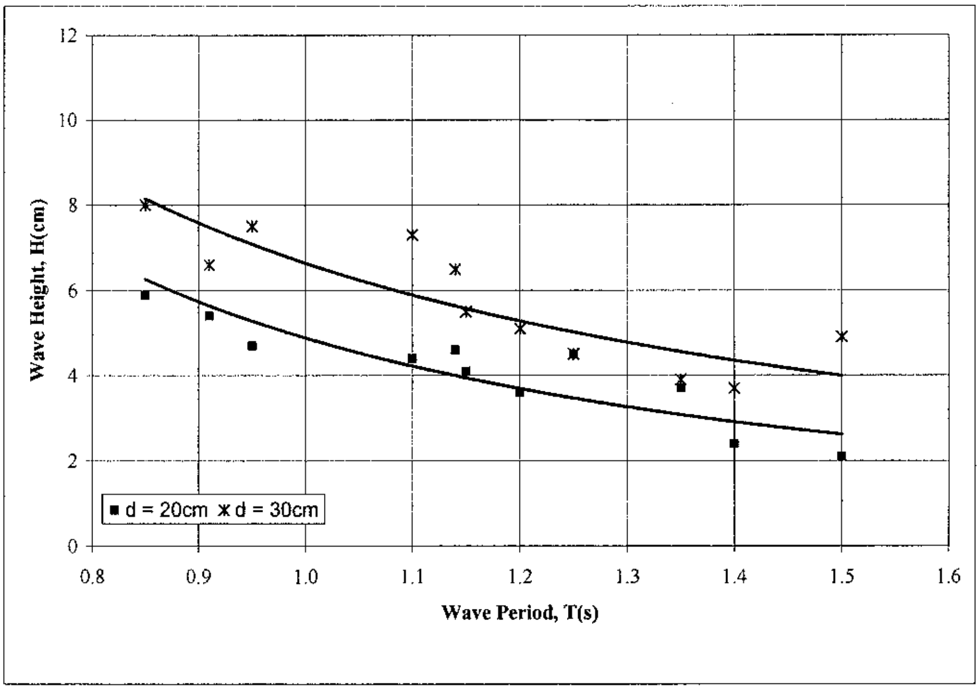


Figure 4.9 Wave Heights for M-4

Shown in the figures above, it can be seen that the relationship between the wave height and the wave period is that as the wave period increases, the wave height decreases. Generally it was observed that the wave heights are higher at water depth of 30cm compared with water depth of 20cm. Therefore it can be seen that the wave heights are not only dependant to the wave period but it is dependant to the water depth as well. It was also observed that the wave heights for all models were relatively the same at the designated wave period. Therefore, it can be concluded that the shapes or the different geometries of the models do not affect the wave heights.

4.7 Determination of Theoretical Wave Forces

The determination of wave height is important as the pressure intensity has a direct proportional value to the maximum wave height. The incident wave heights obtained from an experiment will be applied into the modified Goda's formula of pressure intensity to obtain the theoretical pressure intensity that would be induced onto the floating breakwater. The modified Goda's formula of pressure intensity is as below:

$$p_1 = 0.5(1 + \cos\beta)(\alpha_1 + \cos^2\beta)\gamma H_{max} \qquad (2.8)$$

$$p_2 = \alpha_3 p_1 \qquad (2.9)$$

$$\alpha_1 = 0.6 + 0.5\left(\frac{2kh}{\sinh 2kh}\right)^2 \qquad (2.10)$$

$$\alpha_3 = 1 - D/d\left[1 - 1/\cosh kd\right] \qquad (2.11)$$

$$p = p_1 + p_2/2 \qquad (2.12)$$

Based on this study, the wave applied onto the models were from the flume, and therefore the incident angle of the wave, β, would be taken as zero. Also, the specific weight of the water, γ, would be taken as 10kN/m^3.

Pertaining to the wave heights obtained the theoretical wave forces were calculated (data are tabulated in *Appendix B*). Dimensionless parameters such as $F/0.5\gamma AH_{max}$, $gT^2/2\pi L$ will be used to illustrate graphically in the figures below. Dimensionless parameters are used to get rid of the particular tests conditions. This is to enable the possibility of generalization of the results obtained to reuse with the real conditions outside. Also, dimensionless parameters are used to make the results comparable with a quantity of the same nature. And finally, it is very much useful to get rid of units, where ratio will be used to simplify the results obtained.

The dimensionless parameter, $F/0.5\gamma AH_{max}$ was derived from the formulae from the hydrostatic force. This dimensionless parameter indicates the characteristics of the forces induced by the waves by relating to the hydrostatic forces. Therefore, in theory, the forces are related to the hydrostatic pressure applied to the wetted area of the structure. The dimensionless parameter $gT^2/2\pi L$ on the other hand relates the wave periods to the normal wavelengths and deepwater wavelengths.

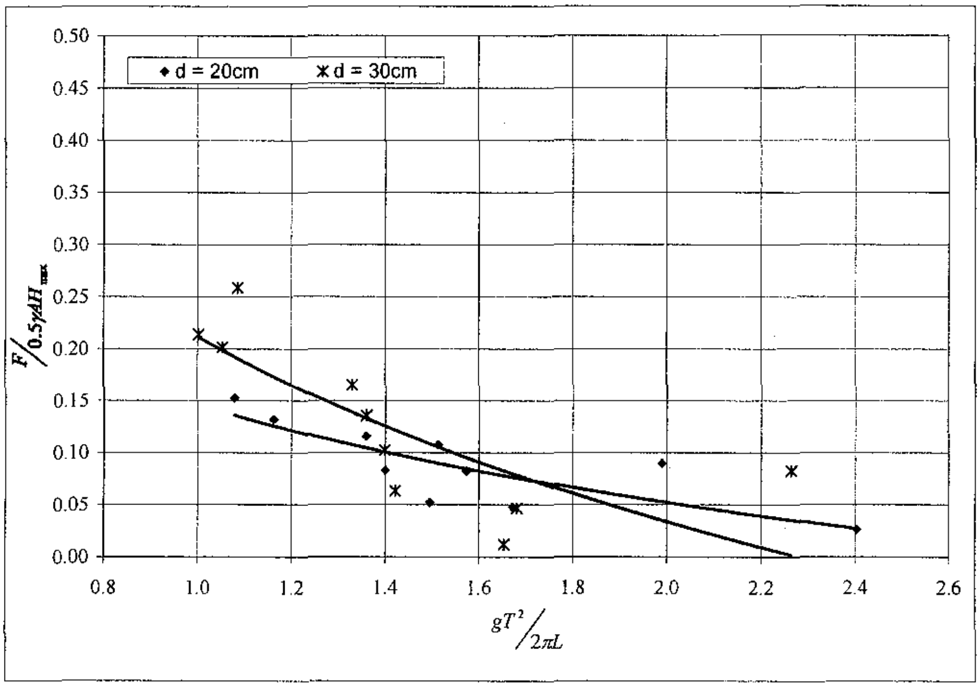


Figure 4.10 Theoretical wave forces applied on M-1

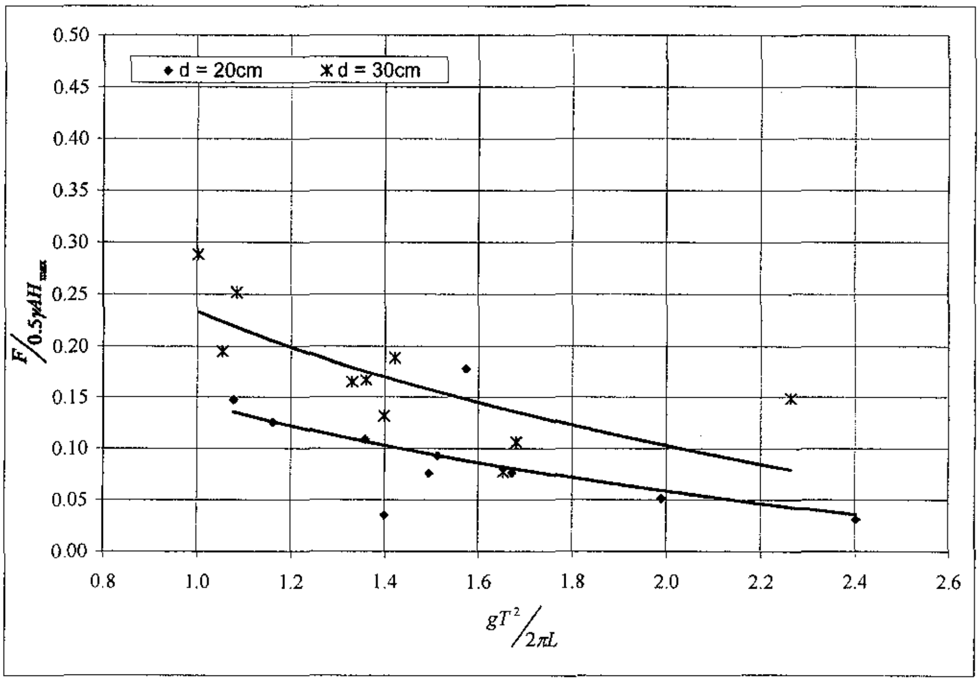


Figure 4.11 Theoretical wave forces applied on M-2

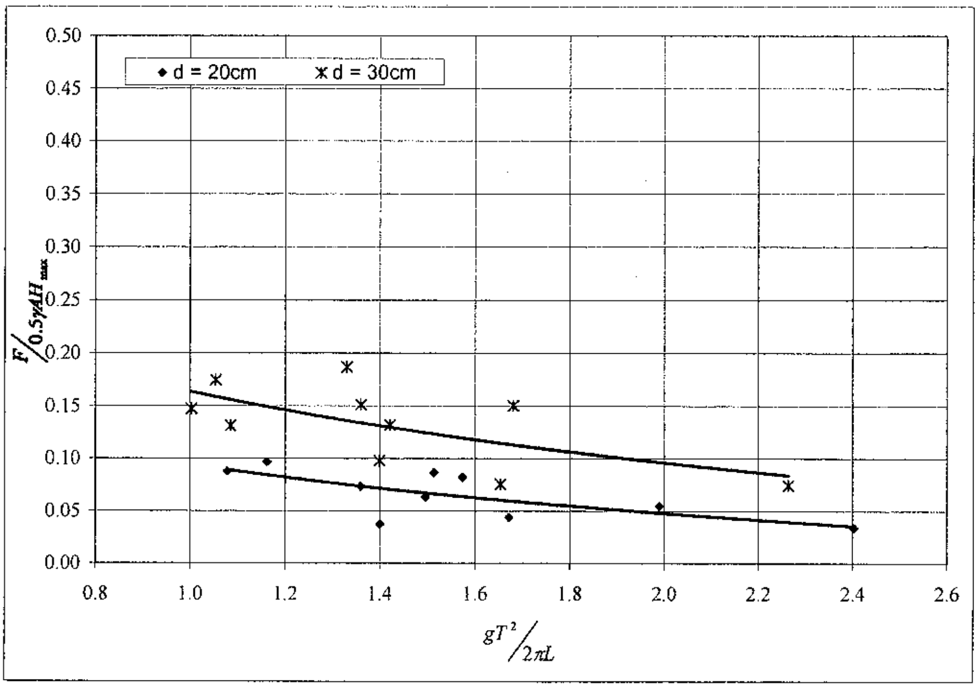


Figure 4.12 Theoretical wave forces applied on M-3

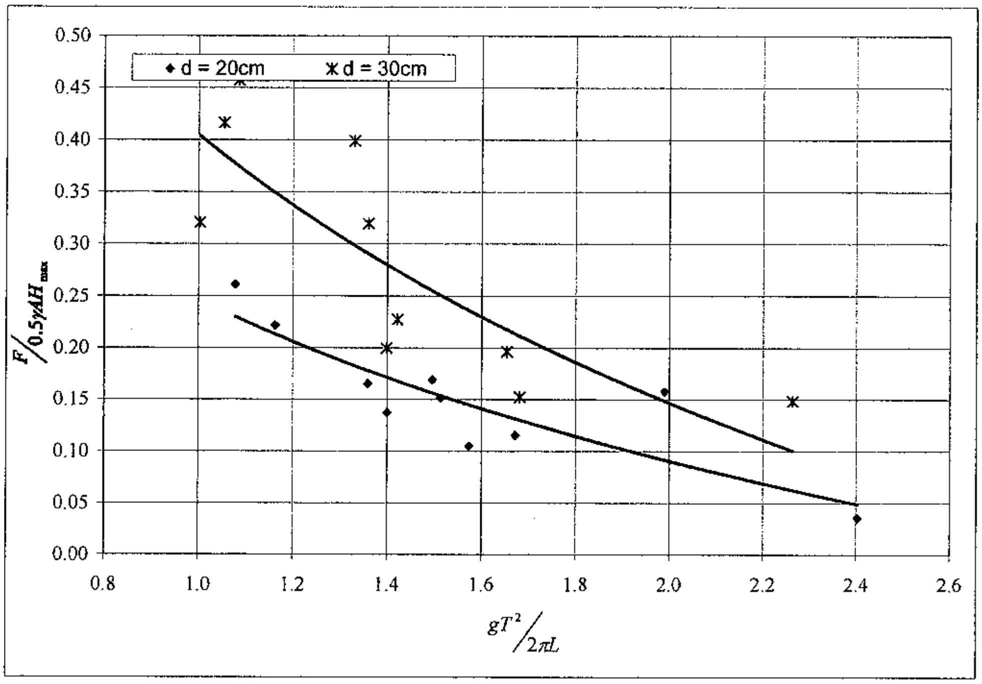


Figure 4.13 Theoretical wave forces applied on M-4

From the figures above, the plots were using the dimensionless parameters $F/0.5\gamma AH_{max}$ versus $gT^2/2\pi L$, where A is the wetted area, H_{max} is the maximum incident wave height, g is the gravitational acceleration in response with two water depths, namely 20cm and 30cm. Best fitted curves were constructed to illustrate the wave forces applied on each models in respect to the water depths. From the plots above, it can be observed that the wave forces have a profile that as the wave period increase, the wave force would decrease as well. Theoretically, it can be seen that the wave forces applied onto M-1 has the least significance between forces in relating to the water depth. The stepped slopped and the inclined face of M-2, M-3 and M-4 shows the wave forces were dissipated by these faces by creating a turbulence beneath the floating breakwater, hence dissipating the energy and therefore creating a difference between the wave forces over the water depths. This would create a relatively constant wave force profiles. It could be seen in the figures above that Model M-4 has the most significant changes and largest theoretical wave forces values. Verification would be done in the following section to confirm such results.

4.8 Calibration of the Cane Pile

As mentioned in Chapter 3, *Methodology II* requires a calibration to obtain a Force vs. Displacement graph of the cane pile to assist in the determination of the experimental wave forces values. The method to obtain this calibration graph is rather simple. The cane pile is subjected to a certain force to obtain the related displacement. This calibration is done in two different depths to correspond to the water depths in this study. The calibrated force versus displacement graph is as below:

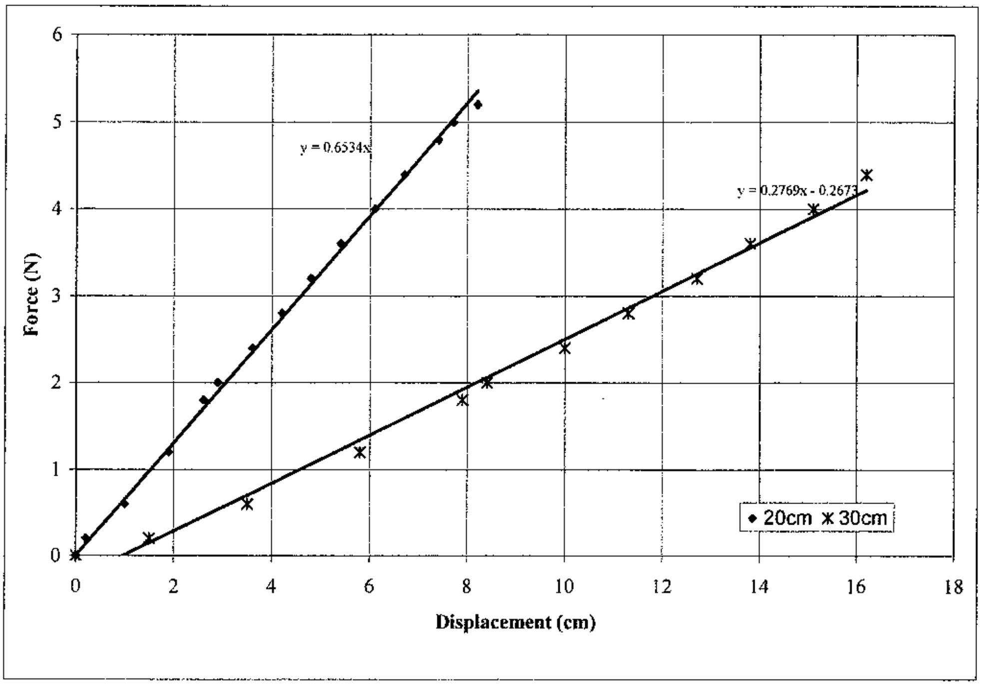


Figure 4.14 Calibration graph for the cane pile for 20cm/30cm water depth

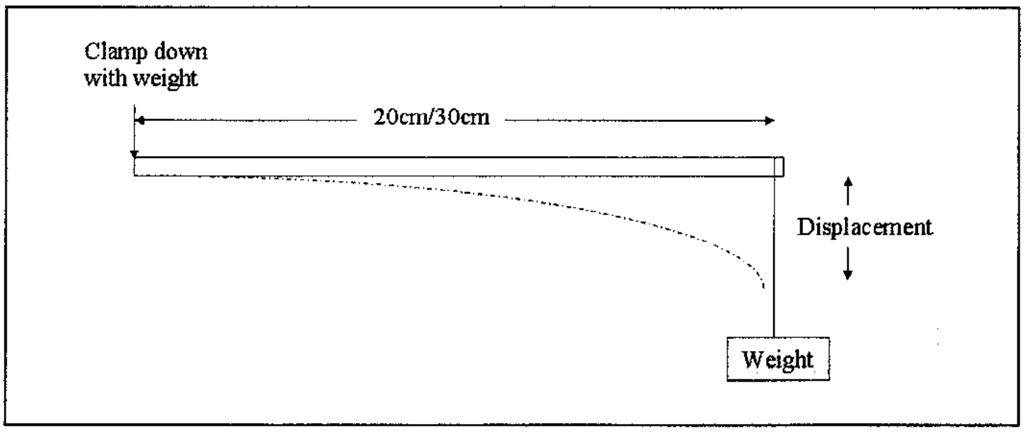


Figure 4.15 Schematic drawing of the calibration experiment

Shown in Figure 4.13 above, the behavior of the can pile was similar to the assumed behavior based on material science. Since the displacement induced by the force is within the elastic limit and the yield point of the cane pile was not reached, the calibration graph should be as shown above. The relationship shown in the graph is similar to Hooke's law. This elastic relationship can be written as:

$$y = 0.6534x \qquad \text{for depth, d} = 20\text{cm} \qquad (4.1)$$

$$y = 0.2769x - 0.2673 \qquad \text{for depth, d} = 30\text{cm} \qquad (4.2)$$

where y is the force and x is the displacement

4.9 Determination of Experimental Wave Forces

The cane pile that was subjected to this simple calibration was then used for the main experimental study in the wave flume. The displacement of the floating breakwater when subjected to wave forces were obtained and applied into *(4.1)* and *(4.2)* to obtain the forces applying onto the body when the body was subjected to wave forces. The data are tabulated in *Appendix B* and graphically illustrated in the figures below:

From the data obtained for all models, namely the M-1, M-2, M-3 and M-4, it was observed that for both water depths of 20cm and 30m, the wave forces applied onto the floating breakwater were increasing with the wave period but some of them peaks when the wave period is at 1.35s. Similar to the theoretical wave forces, the data are tabulated and illustrated in dimensionless parameters. These dimensionless parameters are the same as the ones used in Section 4.7. The wave forces observed showed that there was an increase in wave forces as the wave period increases. This showed that as the wave frequency decreases, the wave force increases. With this, a relationship between the wave force and the wave period can be established. It can be generally accepted that higher waves forces will be induced as the wave period increase, which the wave frequency decreases. Based on the figures below, similar to the wave heights, there was a range between the wave forces at different depths. This shows that the wave forces have a relationship with the water depth as well.

Shown in the Figure 4.16, Model M-1 has the largest increase with the wave period compared with other models. Model M-1 is the control unit that takes the shape of a rectangular box. The rectangular box unit has no stepped or inclined face like the other models, therefore the energy is not dissipated as significantly as the models M-2, M-3, and M-4. Also, the flat surface area at the seaside of the floating breakwater would absorb the wave forces, instead of dissipating the wave forces, and added wave forces would be induced due to the reflected waves. The energy absorbed by M-1 would be the largest as well due to its weight, which is the heaviest among all the models.

The relationship between the wave height and the wave force is simple as shown in Goda's formula, the higher the wave, the larger the wave force. However, the wave height increases with the increase of the wave frequency, which subsequently leading to the decrease of the wave period. Therefore, as shown in Section 4.7, the wave forces decreases with the increase of the wave period. This contradicts the results obtained from the experimental study. This could be due to the difference in draft between the experiment, and the hypothetical theory. In Goda's formula, the draft of the breakwater is the total water depth, because it is a solid breakwater. This would result in a different value between the hypothetical calculations and the experimental values. The hypothetical calculations would yield a larger wave force value due to the much bigger draft. Also, because the total energy of the wave is absorbed by the solid breakwater, the force acting upon it would be much larger than the floating breakwater, which allows no wave energy to transmit across to the lee side of the structure. Therefore, it can be concluded that Goda's formula based on the principles of a conventional breakwater is not suitable to be applied in this study.

Based on the author's observation, the wave forces mostly peak when the wave period is at 1.35s. This shows that the wave period of 1.35s could be the resonant frequency to the floating breakwater. The resonant frequency of the floating breakwater could be due to the material used instead of the geometry of the floating breakwater. This can be seen as all the models were made of the same material, which is wood, but has a different geometry, but has most of them, has a peak wave force at the wave period of 1.35s.

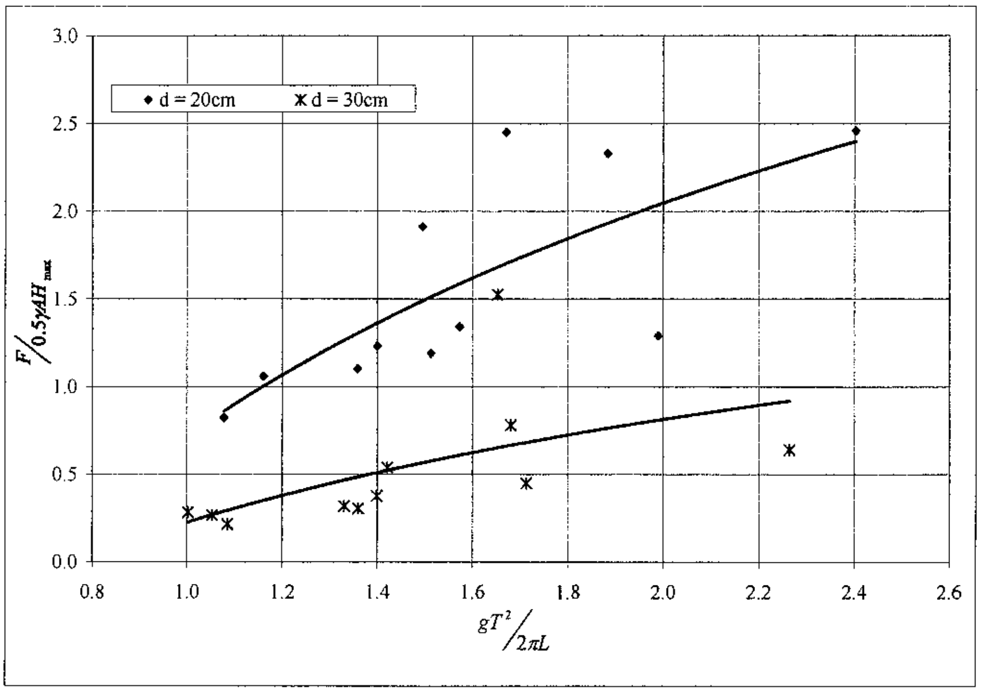


Figure 4.16 Experimental wave forces applied on M-1

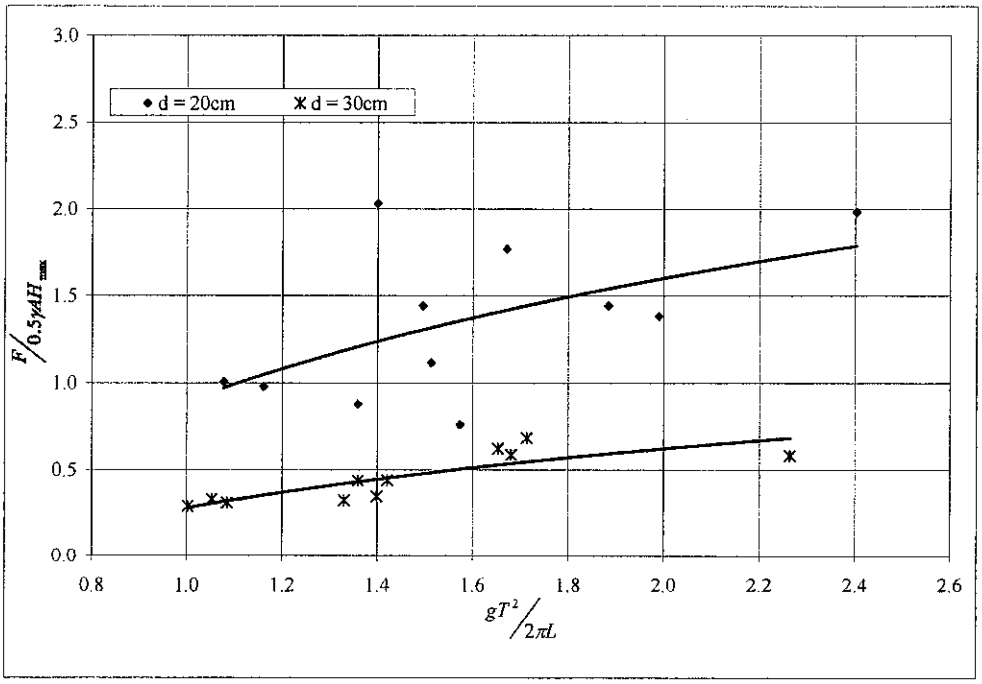


Figure 4.17 Experimental wave forces applied on M-2

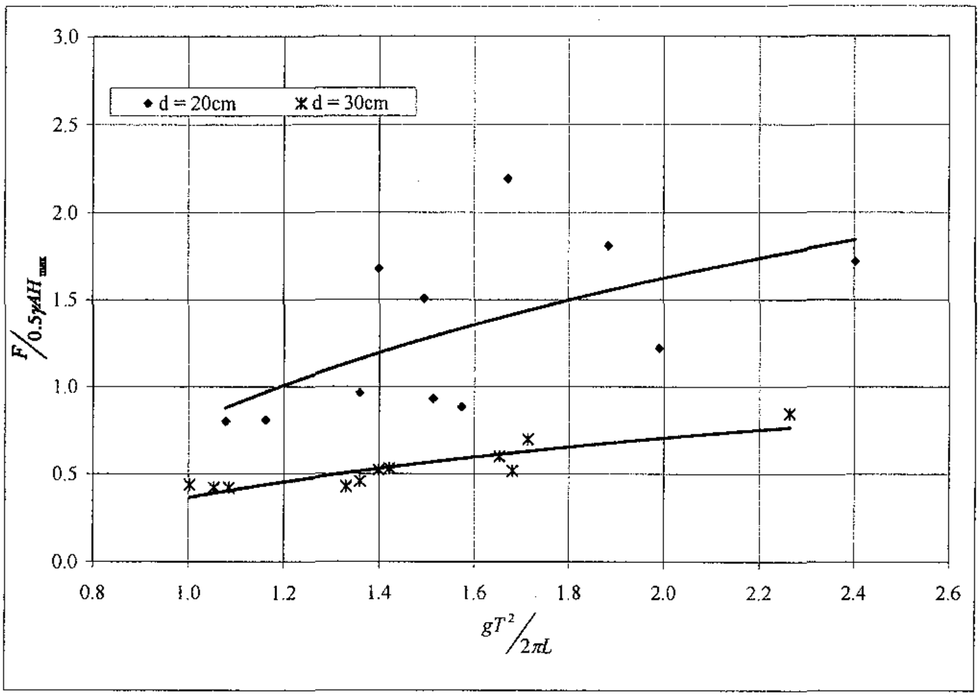


Figure 4.18 Experimental wave forces applied on M-3

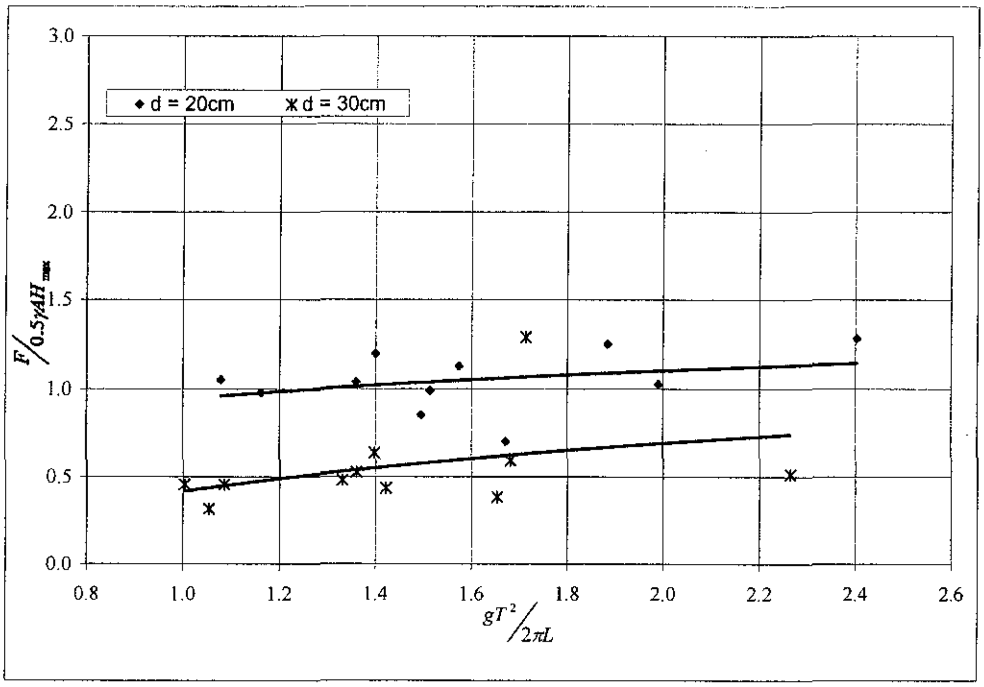


Figure 4.19 Experimental wave forces applied on M-4

As observed in the figures above, the maximum wave force induced on all the models would be on Model M-4, where the wave force is valued 9.58N. This value could be used as the design wave height for the design of these models. This design wave height could be used in the further study of these models.

4.10 Further Study of the Wave Forces

During the experimental study, it was observed that the wave forces peaks when the wave period is at 1.35s for most of tested models. As explained in the previous sections, a presumption was made that the reason of the wave force peaking at the wave period of 1.35s is due to the resonant frequency of the wave coincides with the natural frequency of the material of the floating breakwater. Therefore, a further detailed study was done to observe this critical wave period. This section is to discuss in details about the motions and displacements of the various models with their corresponding wave heights. The displacements and the wave heights are tabulated with the wave cycle at the wave period of 1.35s. The data are tabulated in the tables in *Appendix B*.

Dimensionless parameters will be used to establish a relationship between the parameters in this study. The wave length, *L*, at a certain wave period which corresponds to a certain water depth would be constant. Also, the displacement is horizontal, which is the same with the wave length. Therefore the displacement, x would be rendered dimensionless using the parameter x/L. On the other hand, the wave height and the water depth is a vertical parameter, and the water depth is constant in a particular test, therefore H/d will be used to establish a relationship between the wave heights and the water depth.

4.10.1 Study of the Motions

The forward displacements induced by the wave forces indicate the displacement along the wave, whereas the backward displacement indicates the opposite direction. The incident and attenuated wave heights were taken 1 glass panel of the wave flume, which is 1.2m away from the floating breakwater. This is to ensure that there were no disruptions induced by the reflected wave heights.

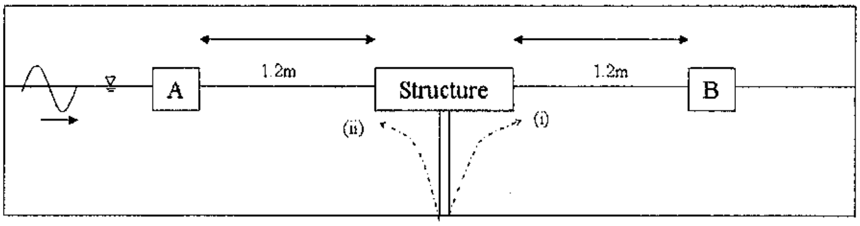
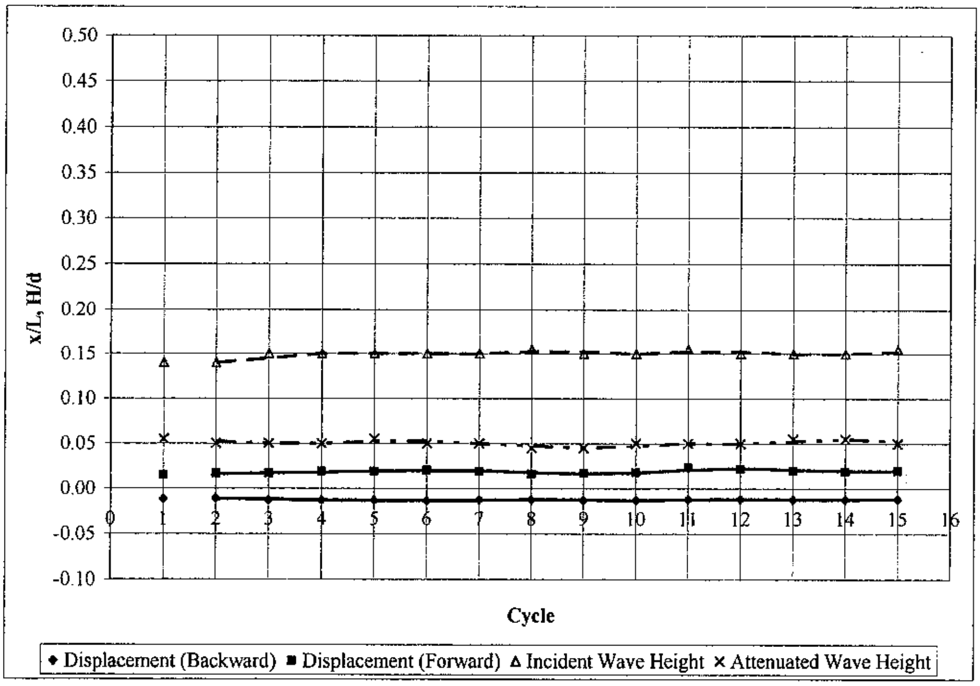


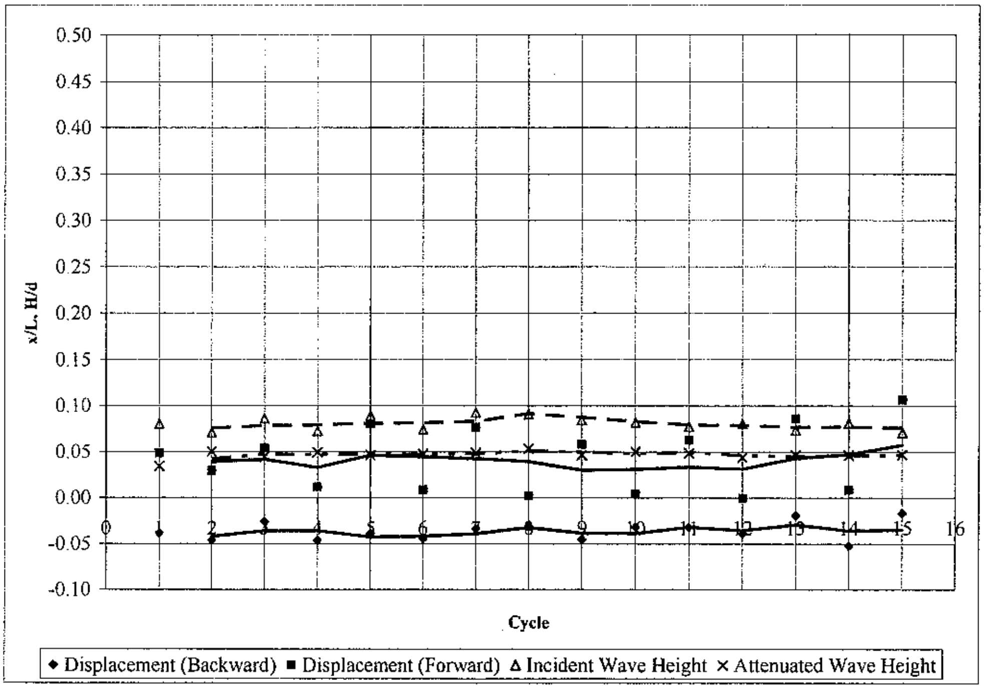
Figure 4.20 Schematic diagram of the detailed study in the wave flume

As shown in Figure 4.20, the incident wave height was measured at location *A* and the attenuated wave height was measured at location *B*. Also shown in the figure, the floating breakwater was piled down t the bottom of the wave flume. The dotted arrows shown in the figure above indicated the displacements of the floating breakwater due to the wave forces induced to it. The dotted arrow facing towards *(i)* is the forward displacement, whereas the dotted arrow facing towards *(ii)* is the backward displacement.

The figures below shows the detailed study of the wave forces at wave period of 1.35s.

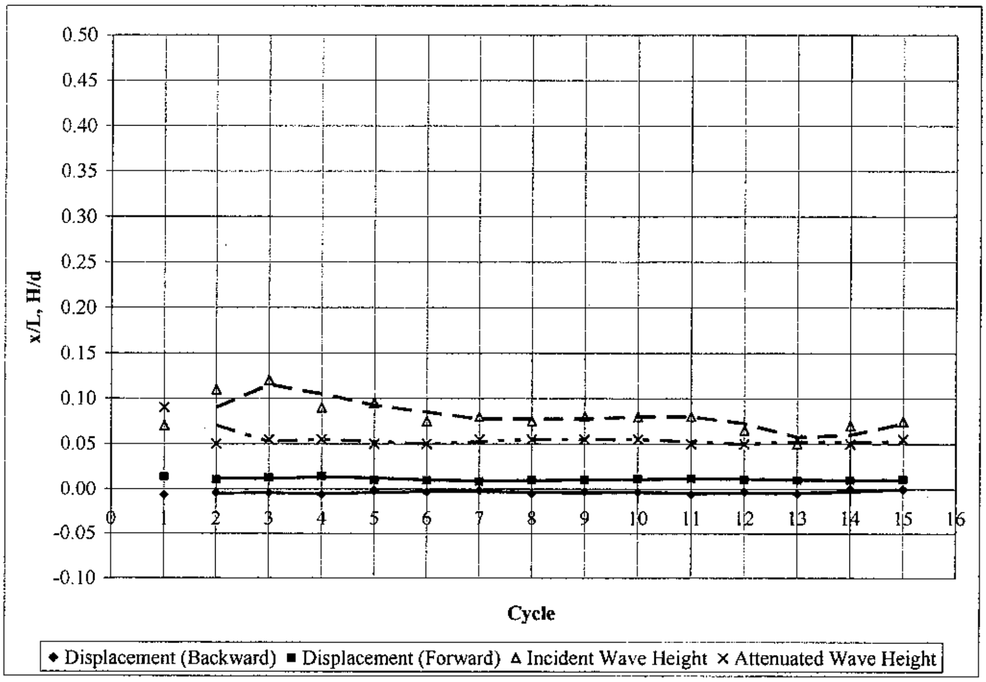


(a) At water depth, d = 20cm

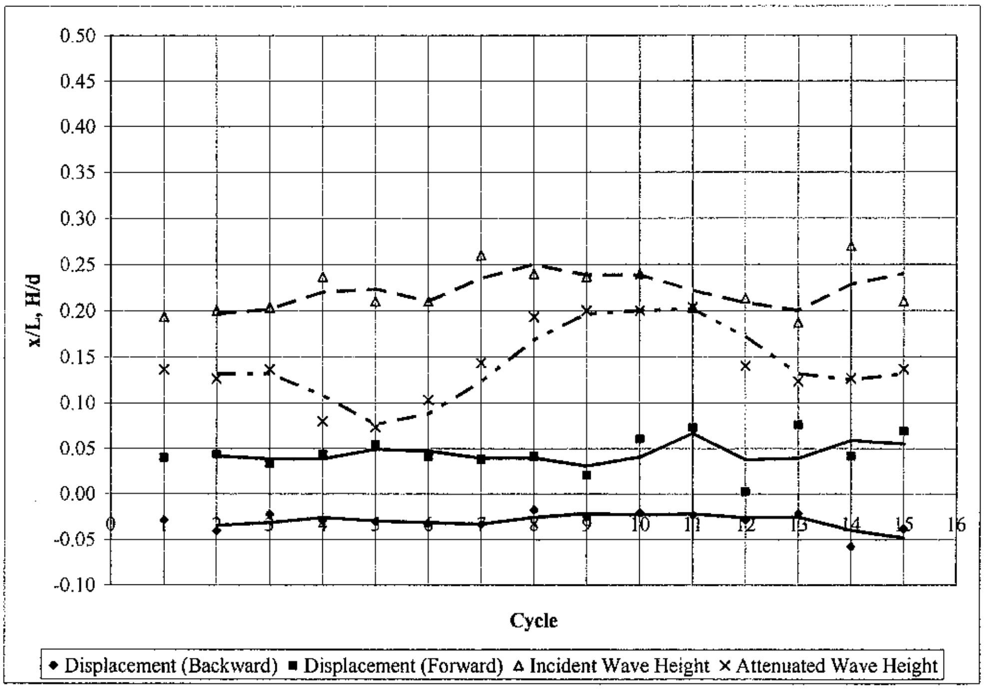


(b) At water depth, d = 30cm

Figure 4.21 Detailed study of the wave forces at T = 1.35s (Model M-1)

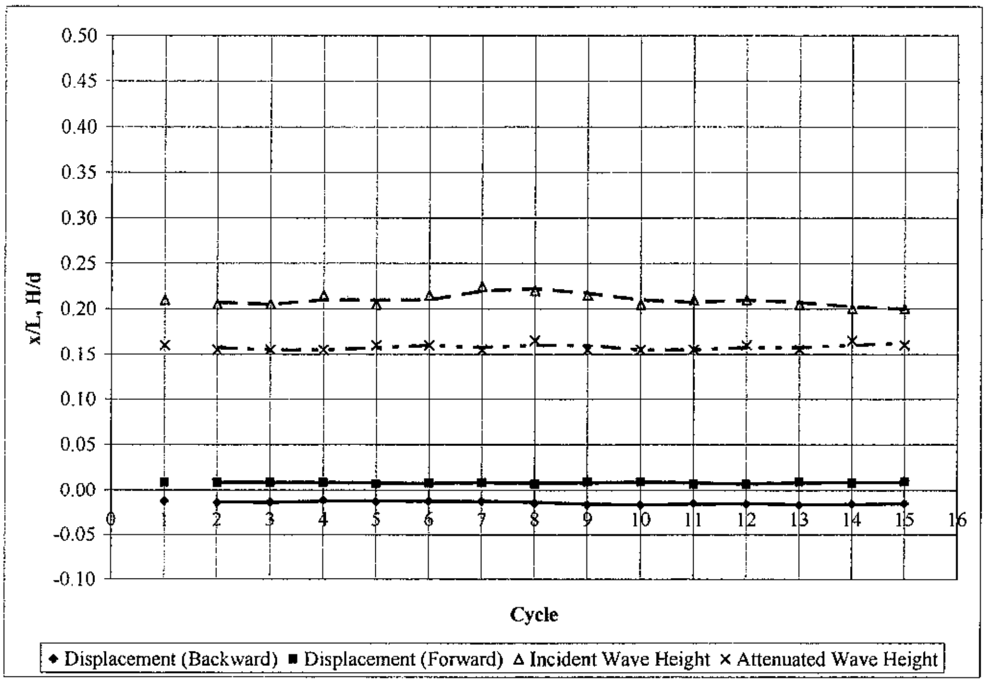


(a) At water depth, d = 20cm

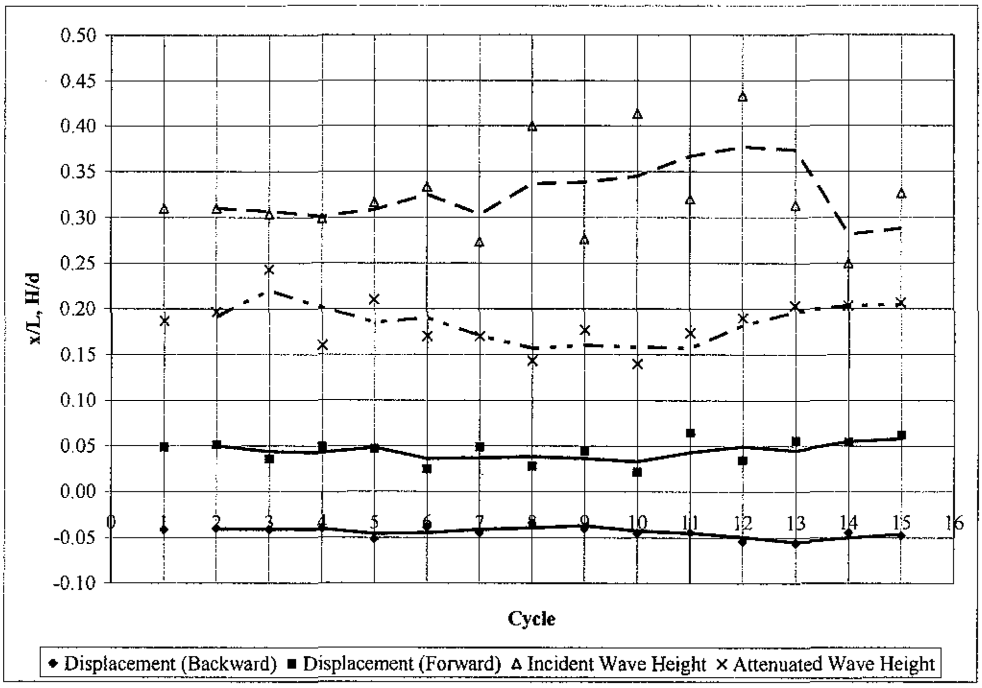


(b) At water depth, d = 30cm

Figure 4.22 Detailed study of the wave forces at T = 1.35s (Model M-2)

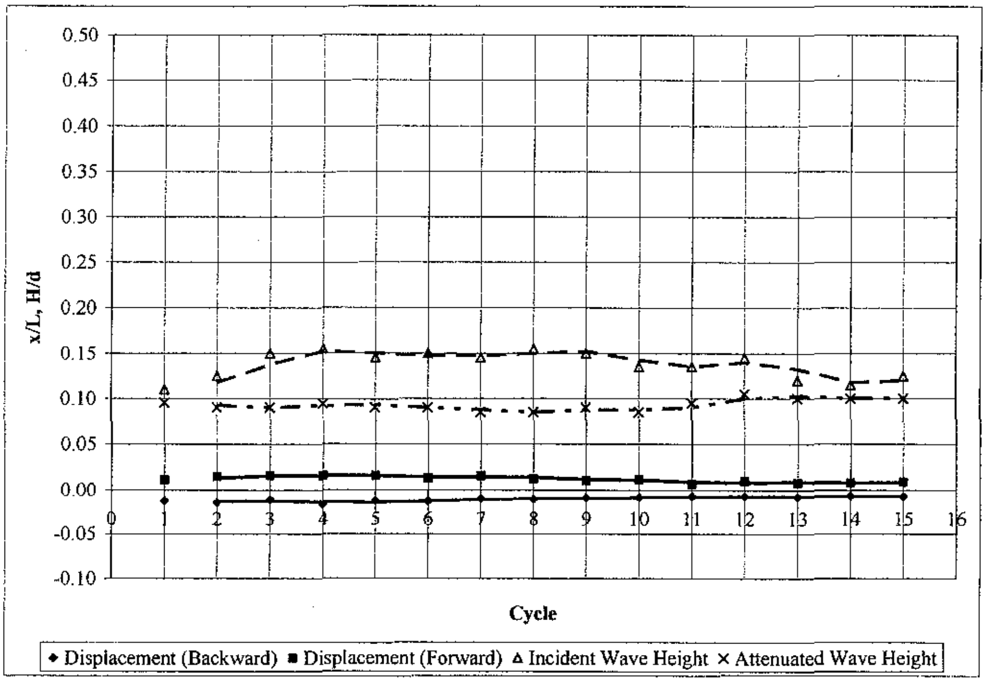


(a) At water depth, d = 20cm

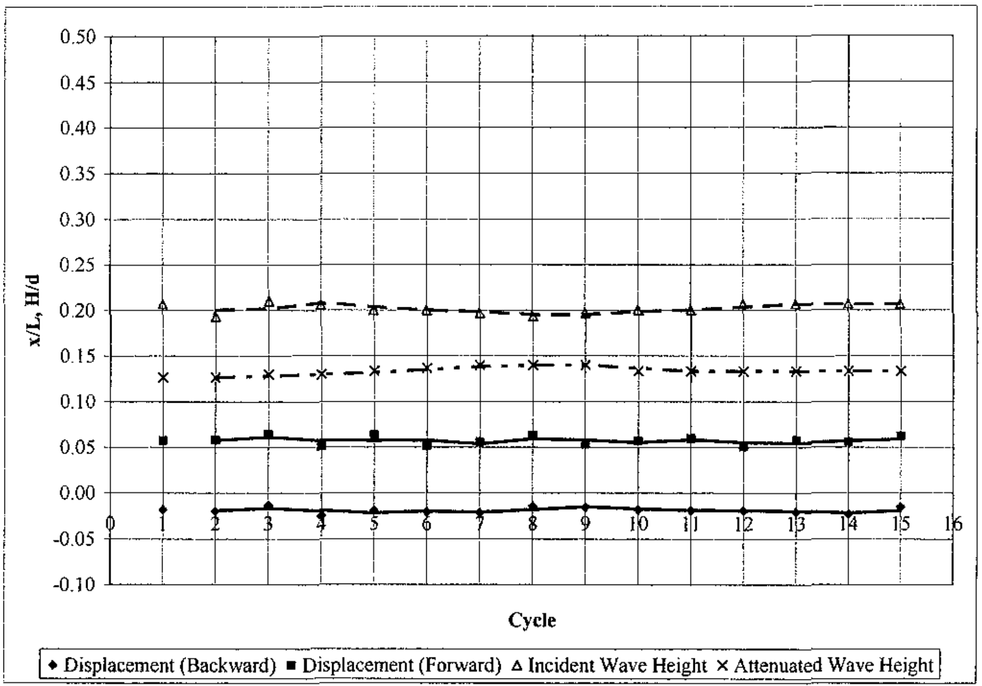


(b) At water depth, d = 30cm

Figure 4.23 Detailed study of the wave forces at T = 1.35s (Model M-3)



(a) At water depth, d = 20cm



(b) At water depth, d = 30cm

Figure 4.24 Detailed study of the wave forces at T = 1.35s (Model M-4)

From the figures above, it was observed that there is a significant difference in displacements between the forward and the backward displacements. The forward displacements are illustrated using positive values whereas the backward displacements are illustrated using negative values. Generally, because the pile used was a flexible material, the wave forces would induce a frontward motion to the floating breakwater, causing the pile to have a forward displacement. Since the displacement of the pile was still within the elastic limit, no permanent deformation would occur and the pile would shift back to its original position. However, it most dynamic motions when a material is subjected to a force, an oscillating motion would occur. Also, because the floating breakwater is in a water body, as the pile returns to its original position, there will be added mass by the water body that causes a reaction force to be induced onto the floating breakwater, rendering a backward displacement. However, as shown in the figures above, the backward displacements are not as high as the forward displacements. Therefore it can be deducted that the wave forces are higher than the reaction forces to the floating breakwater. The justification to this is because the wave energy at the leeside of the structure has been attenuated, and therefore a lower force would be induced to the floating breakwater to cause a smaller displacement.

4.10.2 Wave Attenuation Performance at T = 1.35s

From the figures above, all models manage to attenuate wave heights successfully. There are significant difference between the incident wave heights and the attenuated wave heights. However, the wave attenuated capabilities of most of the models is not that effective at the wave period of 1.35s. The coefficient of transmission, C_t, is used to determine the effectiveness of the wave attenuation capability. It is a measure used to quantify the degree of wave attenuation of breakwaters. The wave attenuation performance for the models is tabulated below:

Table 4.8 Wave attenuation performance for all models at T = 1.8s

Model	Water depth, d(cm)	C_t
M1	20	0.3390
	30	0.5870
M2	20	0.7160
	30	0.6643
M3	20	0.7544
	30	0.5852
M4	20	0.6875
	30	0.6619

Shown in the table above, Model M-1 has the best attenuation performance at wave period, T = 1.35s. The rest of the models have less wave attenuation performance at this particular wave period. This coincides with the previous study indicating the degree of wave attenuation of the M-2, M-3, and M-4 models, which are the WSS models designed by previous studies, increases with the decreasing of the wave period. The wave period of 1.35s is at the higher end of the scope of this study comparing with wave periods such as 0.85s.

4.10.3 Wave Energy

Energy is imparted to a medium when work is done upon it. The total wave energy induced onto the floating structure can be said as $E = (\rho g H^2)/8$, where E is the energy, ρ is the specific gravity of water, g is the gravitational acceleration, and H is the wave height. Therefore, it can be seen that the amount of energy carried by a wave is very much related to the amplitude of the wave. This energy will be spread into four parts as the wave hits the floating breakwater. These four parts are separated into the energy that is transmitted to the leeside of the floating breakwater, the reflected wave energy, the energy absorbed by the pile of the structure, and the dissipated energy.

The energy transmitted to the leeside of the structure can be obtained by the coefficient of transmission as discussed in the section above. Because the wave energy is related to the wave heights, the coefficient of transmission is a good ratio or percentage to indicate how much energy has been transmitted to the leeside of the structure.

The energy reflected by the floating breakwater can also be determined using a coefficient as the coefficient of transmission. This coefficient is known as the coefficient of reflection, C_R. The coefficient of reflection is determined to know the reflective performance of the floating breakwater. Similar to the coefficient of transmission, the ratio or percentage of the reflected wave over the incident wave height would be the percentage of the wave energy being reflected back to the sea. Therefore, using the coefficient of reflection could determine the reflected wave energy.

The energy of a structure can simply be defined as the capacity of the structure to do work. Forces are imparted onto the structure would do work onto it, giving it kinetic energy as well as potential energy. The idea is as such, as the wave forces are applied onto the structure, some initial displacement would be introduced. The displacement is due to some force introduced to the body to displace it from a given amount from rest. The more force is applied to the body, the more displacement it would is given to it, and the displacement is directly proportionate to the energy imparting onto the body. The energy absorbed by the structure could also be known as the strain energy, where the energy that is caused by deformation of the structure (Kassimali, 2005). The relationship between the work and strain energy of a structure is based on the principle of conservation of energy, which means that the work performed on a structure by external applied forces is equal to the internal energy. Therefore, the expression for the strain energy induced or the energy absorbed by the pile by the wave energy is as below:

$$U = Fx \tag{4.3}$$

Where U is the internal strain energy

F is the cross sectional properties the beam

x is the displacement induced by the force

Based on the equation above, the displacement induced by the force, which subsequently the strain energy of the pile, can be defined as above. Therefore, by multiplying the displacement which was induced by the wave force with the wave force, the strain energy, or the energy absorbed by the structure, can be obtained.

The wave energy that is dissipated could be due to turbulence underneath the floating breakwater, energy to overcome water friction, sound energy, etc. The wave energy that was dissipated away could not be measured directly. Therefore, the wave energy that is dissipated is the remaining of the wave energy after taking into account the energy that was reflected, transmitted to the leeside of the structure, and the energy that was absorbed by the structure.

The calculations of distributing the wave energy are tabulated in *Appendix B* and are graphically illustrated in the figures below:

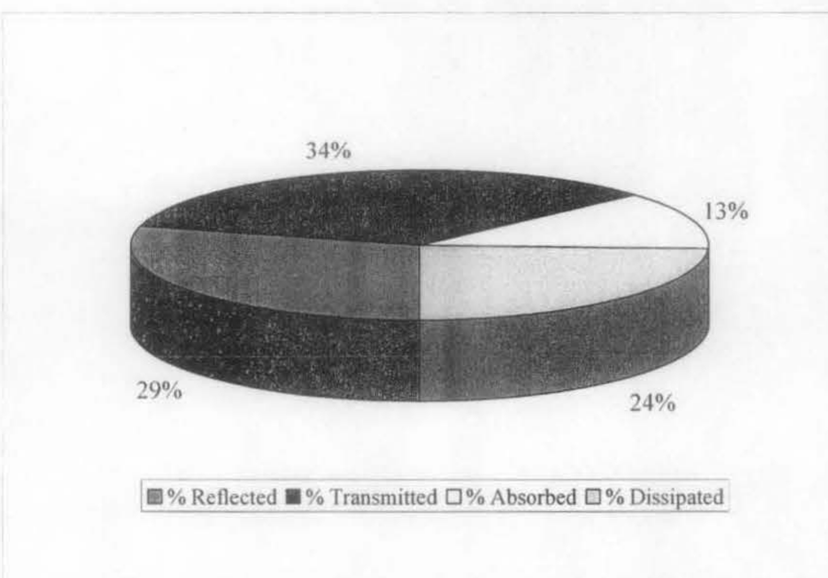


Figure 4.25 Energy distributions for Model M-1, 20cm water depth

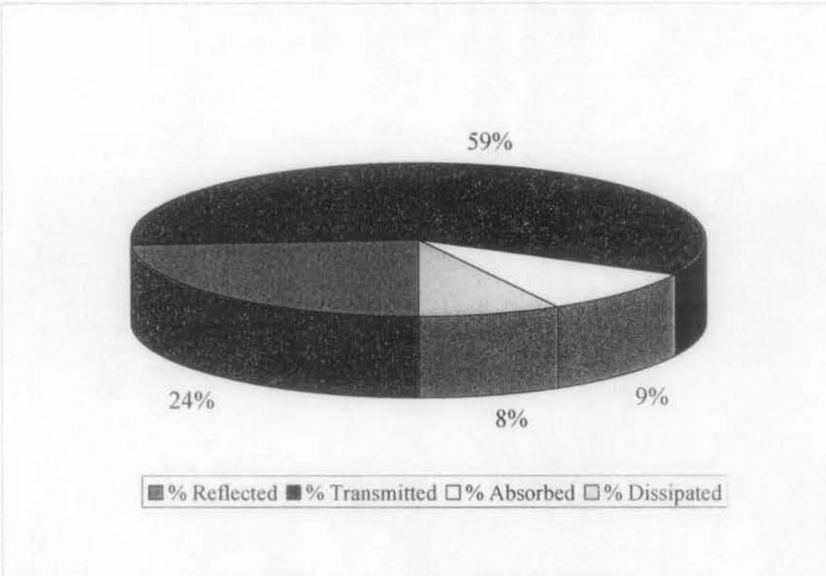


Figure 4.26 Energy distributions for Model M-1, 30cm water depth

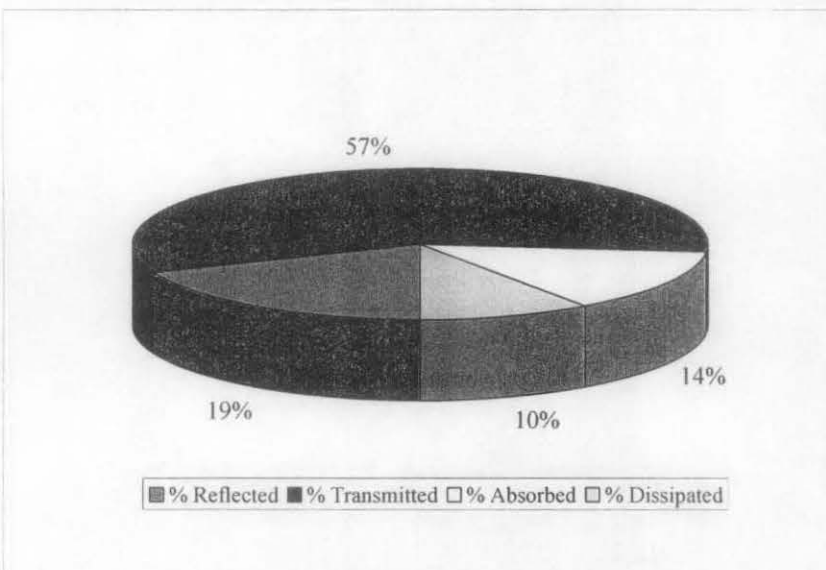


Figure 4.27 Energy distributions for Model M-2, 20cm water depth

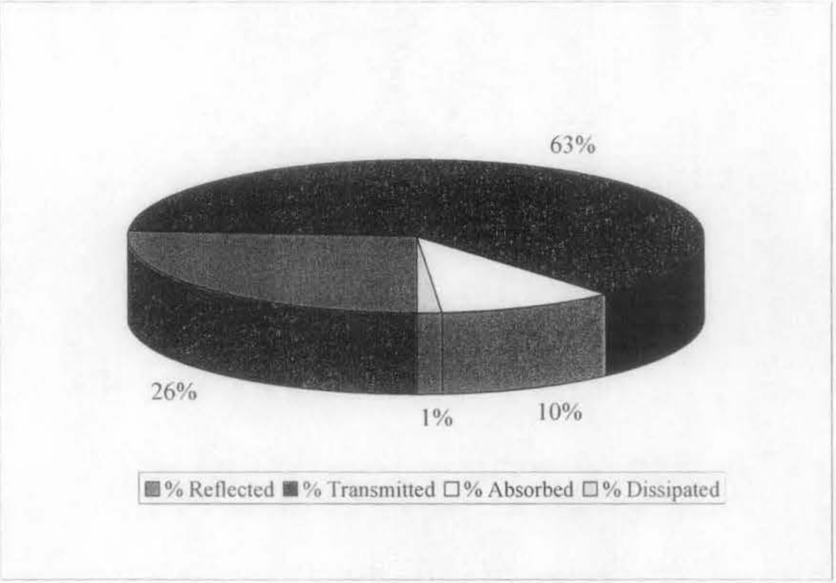


Figure 4.28 Energy distributions for Model M-2, 30cm water depth

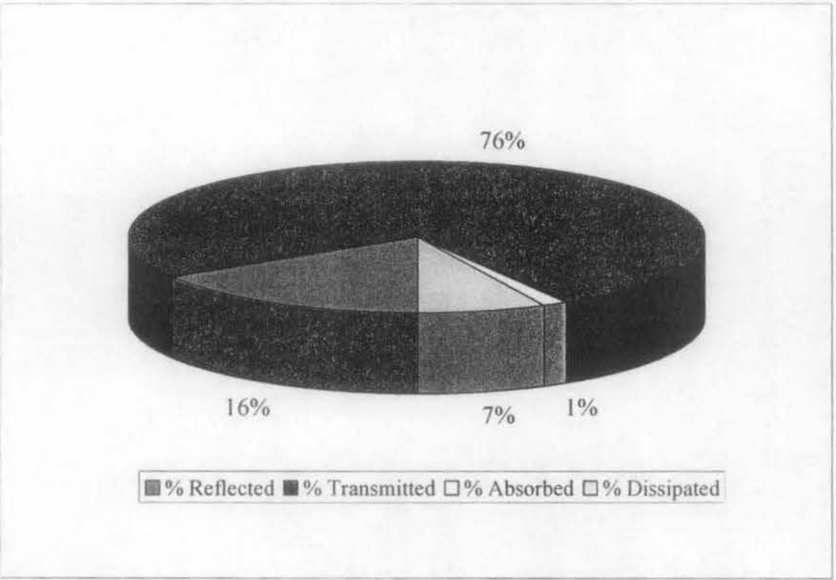


Figure 4.29 Energy distributions for Model M-3, 20cm water depth

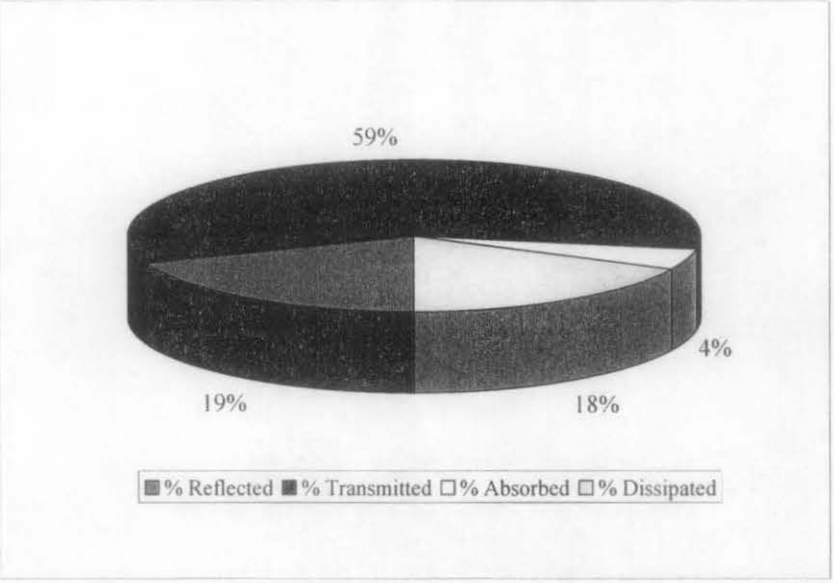


Figure 4.30 Energy distributions for Model M-3, 30cm water depth

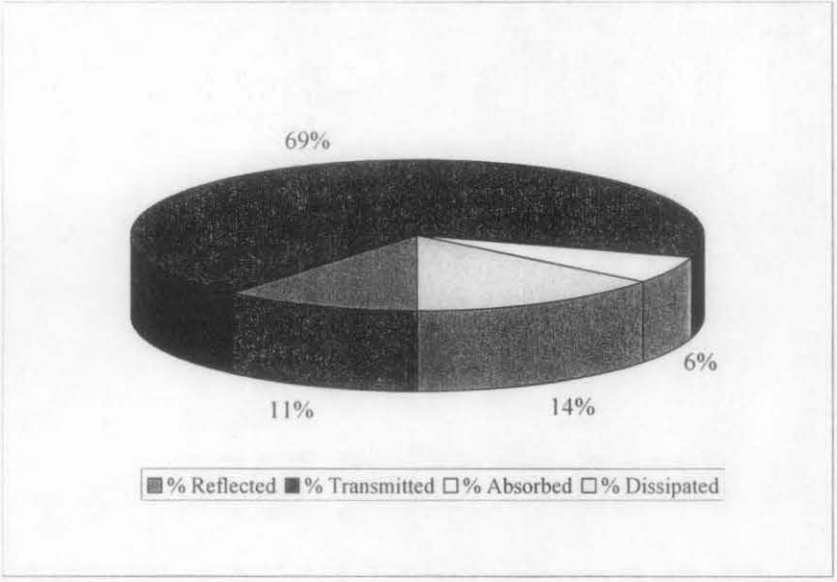


Figure 4.31 Energy distributions for Model M-4, 20cm water depth

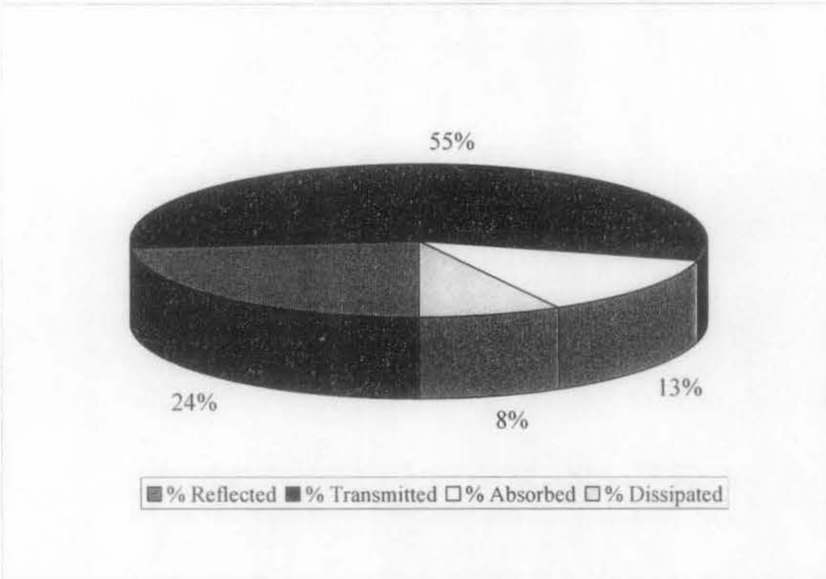


Figure 4.32 Energy distributions for Model M-4, 30cm water depth

As it can be seen in the figures above, the poorest performance among the models in this study would to be Model M-3 at water depth of 20cm as it has the largest value of energy transmitted to the leeside of the breakwater. However, it must be noted that this distribution is done only for the wave period of 1.35s, where most of the wave force peaks.

All percentages of wave energy are correspondent to water depths 20cm and 30 respectively.

Model M-1 transmitted majority of the energy, which is 54% and 59% of the wave energy Model M-1 also reflected majority of the wave energy, valuing at 29% and 24% of the wave energy. This could be due to the flat wetted surface where it could act like a vertical wall. Model M-1 is a poor absorbent, only absorbing 6% and 16% of the wave energy. The other 24% and 8% of the wave energy was dissipated away through various mediums.

Model M-2 has also poor wave attenuation performance as 57% and 63% of the wave energy was transmitted to the leeside of the. Similar to Model M-1, it reflected majority of the wave energy, with a value of 19% and 26%. Model M-2 absorded only a very small portion of the wave energy, which is 10% and 1% of the total wave energy. The rest of the 14% and 10%of wave energy were dissipated away.

Model M-3 has the poorest wave attenuation performance among all models at wave period of 1.35s. In total, the model allowed 76% and 59% of the wave energy to be transmitted to the leeside of the structure. Similar to the models M-1 and M-2, it absorbed very little of the wave energy, which are 1% and 4% of the wave energy. The other 7% and 18% of the wave energy were dissipated.

Model M-4 also has rather poor wave attenuation performance with 69% and 55% of the wave energy dissipated to the leeside of the structure. Again, the structure has poor absorbent performance, having only 6% and 13% of the wave energy being absorbed. However, it shows that the structure has good reflecting performance as it reflected 11% and 24% of the wave energy. The rest of the 14% and 8% wave energy was dissipated by various mediums.

Based on the energy analysis, the distribution shows that the average energy transmitted for all models are about 50%. Therefore this shows that all models have poor performance at the wave period of 1.35s. Also observed in the figures above, much percentage of the wave energy was reflected by the structure. However, the high values of wave forces applied onto the floating breakwater in the experiments as mentioned in the sections above does not coincide in the analysis as the models absorbed only a very small portion of the wave energy. The average percentages of absorbed energy for all models are relatively low, with a value of 9%. Therefore it can be deducted that majority of the wave forces applied onto the models wre reflected and transmitted over to the leeside of the floating breakwater. Also based on observation, the models are relatively more effective in attenuating wave energies at deeper water depths. This is because the models tend to reflect the wave energy at deeper water depths.

4.10.4 Discussions

Based on calculations done (data tabulated in *Appendix B*), the standard statistic of this detailed study would show the standard deviation and the coefficient of variation of all the data obtained. The tables below are a summary of the standard statistics of these data.

Table 4.9 Standard statistics of forward displacement

Model	Water depth, d (cm)	Standard Deviation	Coefficient of Variation
M-1	20	0.365	0.113
	30	4.316	0.807
M-2	20	0.257	0.133
	30	2.374	0.417
M-3	20	0.153	0.107
	30	1.596	0.284
M-4	20	0.153	0.077
	30	1.596	0.221

Table 4.10 Standard statistics of backward displacement

Model	Water depth, d (cm)	Standard Deviation	Coefficient of Variation
M-1	20	0.124	0.059
	30	1.256	0.277
M-2	20	0.347	0.559
	30	1.229	0.326
M-3	20	0.278	0.111
	30	0.734	0.132
M-4	20	0.278	0.158
	30	0.734	0.309

From the table above, the coefficient of variation represents the ratio of standard deviation over the mean of the data. This coefficient is useful for statistic to compare the deviation from one data to another. In other words, the lower the ratios of standard deviation to the mean, the better the data series are. Therefore, from the table above, the coefficients of variation for majority of the data are relatively low, ranging from 10% to 30%. However, there are some cases where the deviation of the data series is high, showing that the data series are not reliable, in this case, the standard statistic for Model M-1 at d = 30cm, and Model M-2 at d = 20cm. Nevertheless, out of 16 data series obtained, only two are not reliable would render the data series usable. In other words, the data series obtained throughout this detailed tests are still very much reliable.

CHAPTER 5

CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

The main purpose of this project is to further the studies made by previous projects that only produce results regarding the wave attenuation performances. From this study, it can be observed that the wave profile is generally increasing with the wave period, which subsequently leads to the decrease of the wave frequency. However, due to the contradicted results obtained from Goda's formula for designing a breakwater, it can be deducted that the formula is rendered unfit for this study.

The conclusion of this project can be summarized as below:

- From this study, the wave force trends obtained could be used to help to select a proper material based on its strength to overcome the applied wave forces

- The models were remade using a different material.

- All models were stable during the whole process of this study.

- Determination of the wavelength indicated that the study was made in transitional water condition.

- Generally the wave forces increase with the wave period. It was observed that the wave forces could have a relationship with the wave period instead of the wave height. However, further detailed studies should be done as the accuracy of this study is not high due to the vague methodology because the equipments were not available for use during this study.

- It was observed that the highest wave force was of Model M-4 with the value of 9.58N. This value could be used as the design wave force for the WSS models.

- It was observed as well most of the models, the wave forces peaks at wave period T = 1.35s. This could be due to the resonant frequency of the material and the waves. However, further studies should be made for a clarification of this hypothetical statement.

- At wave period of 1.35s, the attenuation performance was poor for the models M-2, M-3, and M-4 (which are the WSS models). This coincides with previous studies that the wave attenuation of the WSS models increases with the wave heights (which decrease with the wave period).

- Goda's formula from the Principle for Breakwater Design is not applicable in this study.

5.2 Recommendations

For further improvement of this study, few recommendations are highlighted below:

- The accuracy of this study is very vague as no proper equipment was used. Better equipments should be prepared and be operational when used for better accuracy.

- Other wave climates and water conditions should be used to evaluate the wave forces applied onto the models to understand the wave forces profile better.

- A further study could be made using cable mooring system instead of the piling system to evaluate the wave forces.

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APPENDIX A

Summary of Tests with Corresponding Values

A: 1 Summary of Main Tests with Corresponding Values

A: 2 Summary of Detailed Tests with Corresponding Values

D	Draft	**L**	wavelength	**$F/(0.5\gamma AH)$**	Dimensionless parameter
T	Wave period	**k**	Wave number	**$gT^2/2\pi L$**	Dimensionless parameter
Hi	Incident wave height	**Ft**	Theoretical wave force		
d	Water depth	**Fe**	Experimental wave force		

Test	Model	D (cm) Measured	d (cm) Measured	T(s) Measured	Lo Calculated	d/Lo Calculated	d/L SPM table	L (m) Calculated	L (m) Measured	Hi (cm) Measured	Ft (N) Calculated	$F/(0.5\gamma AH)_t$	Fe (N) Measured	$F/(0.5\gamma AH)_e$	$gT^2/2\pi L$ =Lo/L
M1-20-01			20	0.85	1.1280	0.1773	0.1558	1.2834	1.047	7.8	21.745	0.153	5.772	0.822	1.078
M1-20-02			20	0.91	1.2929	0.1547	0.1872	1.0682	1.113	7.2	20.393	0.132	6.861	1.059	1.161
M1-20-03			20	0.95	1.4091	0.1419	0.1765	1.1330	1.037	6.8	18.914	0.116	6.752	0.103	1.359
M1-20-04			20	1.10	1.8892	0.1059	0.1463	1.3670	1.249	6.4	18.683	0.108	6.861	1.191	1.513
M1-20-05	M-1	6.0	20	1.14	2.0291	0.0986	0.1399	1.4298	1.020	6.0	16.619	0.090	6.970	1.291	1.990
M1-20-06			20	1.15	2.0648	0.0969	0.1385	1.4438	1.475	5.5	16.770	0.083	6.098	1.232	1.400
M1-20-07			20	1.20	2.2483	0.0890	0.1312	1.5244	1.429	5.5	16.634	0.082	6.643	1.342	1.573
M1-20-08			20	1.25	2.4395	0.0820	0.1320	1.5152	1.632	4.3	13.440	0.052	7.405	1.914	1.495
M1-20-09			20	1.35	2.8455	0.0703	0.1143	1.7498	1.703	4.1	12.944	0.048	9.039	2.450	1.671
M1-20-10			20	1.50	3.5129	0.0569	0.1011	1.9785	1.462	3.1	9.431	0.026	6.861	2.459	2.403
M1-30-01			30	0.85	1.1280	0.2659	0.2823	1.0627	1.040	10.4	27.638	0.259	2.004	0.214	1.085
M1-30-02			30	0.91	1.2929	0.2320	0.2526	1.1876	1.289	9.3	25.518	0.214	2.373	0.284	1.003
M1-30-03			30	0.95	1.4091	0.2129	0.2363	1.2695	1.337	9.0	24.861	0.201	2.142	0.265	1.054
M1-30-04			30	1.10	1.8892	0.1588	0.1909	1.5713	1.420	8.1	22.638	0.165	2.327	0.319	1.330
M1-30-05	M-1	6.0	30	1.14	2.0291	0.1479	0.1813	1.6545	1.493	7.3	20.613	0.135	2.004	0.305	1.360
M1-30-06			30	1.15	2.0648	0.1453	0.1792	1.6737	1.453	5.0	14.039	0.063	2.419	0.538	1.421
M1-30-07			30	1.20	2.2483	0.1334	0.1697	1.7676	1.608	6.3	18.081	0.103	2.142	0.378	1.399
M1-30-08			30	1.25	2.4395	0.1230	0.1607	1.8668	1.452	4.3	12.072	0.047	3.019	0.780	1.680
M1-30-09			30	1.35	2.8455	0.1054	0.1459	2.0568	1.257	5.8	15.844	0.083	0.342	0.640	2.264
M1-30-10			30	1.50	3.5129	0.0854	0.1283	2.3379	2.125	2.1	6.435	0.012	2.881	1.524	1.653
M2-20-01	M-2	6.5	20	0.85	1.1280	0.1773	0.1558	1.2834	1.047	7.1	21.314	0.148	6.970	1.007	1.078
M2-20-02			20	0.91	1.2929	0.1547	0.1872	1.0682	1.113	6.5	19.834	0.126	6.207	0.980	1.161
M2-20-03			20	0.95	1.4091	0.1419	0.1765	1.1330	1.037	6.1	18.269	0.109	5.227	0.879	1.359
M2-20-04			20	1.10	1.8892	0.1059	0.1463	1.3670	1.249	5.5	17.311	0.093	5.990	1.117	1.513
M2-20-05			20	1.14	2.0291	0.0986	0.1399	1.4298	1.020	4.2	12.524	0.051	5.628	1.383	1.990
M2-20-06			20	1.15	2.0648	0.0969	0.1385	1.4438	1.475	3.3	10.860	0.039	6.534	2.031	1.400
M2-20-07			20	1.20	2.2483	0.0890	0.1312	1.5244	1.429	7.5	24.479	0.179	5.554	0.760	1.573
M2-20-08			20	1.25	2.4395	0.0820	0.1320	1.5152	1.632	4.8	16.203	0.076	6.752	1.443	1.495

M2-20-10			20	1.50	3.5129	0.0569	0.1011	1.9785	1.462	3.1	10.179	0.031	5.990	1.982	2.403
M2-30-01			30	0.85	1.1280	0.2659	0.2823	1.0627	1.040	9.5	27.183	0.252	2.881	0.311	1.085
M2-30-02			30	0.91	1.2929	0.2320	0.2526	1.1876	1.289	10.0	29.578	0.288	2.834	0.291	1.003
M2-30-03			30	0.95	1.4091	0.2129	0.2363	1.2695	1.337	8.2	24.421	0.195	2.650	0.331	1.054
M2-30-04			30	1.10	1.8892	0.1588	0.1909	1.5713	1.420	7.5	22.606	0.165	2.373	0.325	1.330
M2-30-05	M-2	6.5	30	1.14	2.0291	0.1479	0.1813	1.6545	1.493	7.5	22.847	0.167	3.204	0.438	1.360
M2-30-06			30	1.15	2.0648	0.1453	0.1792	1.6737	1.453	8.0	24.230	0.189	2.434	0.440	1.421
M2-30-07			30	1.20	2.2483	0.1334	0.1697	1.7676	1.608	6.6	20.442	0.132	2.234	0.347	1.399
M2-30-08			30	1.25	2.4395	0.1230	0.1607	1.8668	1.452	6.0	18.169	0.106	3.434	0.587	1.680
M2-30-09			30	1.35	2.8455	0.1054	0.1459	2.0568	1.257	7.2	21.199	0.149	4.080	0.581	2.264
M2-30-10			30	1.50	3.5129	0.0854	0.1283	2.3379	2.125	4.9	16.225	0.078	2.973	0.622	1.653
M3-20-01			20	0.85	1.1280	0.1773	0.1558	1.2834	1.047	7.2	16.609	0.088	4.247	0.803	1.078
M3-20-02			20	0.91	1.2929	0.1547	0.1872	1.0682	1.113	7.5	17.559	0.097	4.465	0.801	1.161
M3-20-03			20	0.95	1.4091	0.1419	0.1765	1.1330	1.037	6.6	15.193	0.074	4.683	0.965	1.359
M3-20-04			20	1.10	1.8892	0.1059	0.1463	1.3670	1.249	7.0	16.860	0.087	4.792	0.931	1.513
M3-20-05	M-3	4.9	20	1.14	2.0291	0.0986	0.1399	1.4298	1.020	5.7	13.070	0.055	5.118	1.222	1.990
M3-20-06			20	1.15	2.0648	0.0969	0.1385	1.4438	1.475	4.5	11.295	0.037	5.554	1.697	1.400
M3-20-07			20	1.20	2.2483	0.0890	0.1312	1.5244	1.429	6.7	16.687	0.082	4.365	0.885	1.573
M3-20-08			20	1.25	2.4395	0.0820	0.1320	1.5152	1.632	5.8	14.905	0.064	6.425	1.507	1.495
M3-20-09			20	1.35	2.8455	0.0703	0.1143	1.7498	1.703	4.8	12.454	0.044	7.732	2.149	1.671
M3-20-10			20	1.50	3.5129	0.0569	0.1011	1.9785	1.462	4.3	10.769	0.034	5.445	1.772	2.403
M3-30-01			30	0.85	1.1280	0.2659	0.2823	1.0627	1.040	9.0	19.795	0.131	2.788	0.422	1.085
M3-30-02			30	0.91	1.2929	0.2320	0.2526	1.1876	1.289	9.4	21.293	0.147	3.019	0.437	1.003
M3-30-03			30	0.95	1.4091	0.2129	0.2363	1.2695	1.337	10.2	23.250	0.174	3.157	0.421	1.054
M3-30-04			30	1.10	1.8892	0.1588	0.1909	1.5713	1.420	10.5	24.199	0.187	3.296	0.427	1.330
M3-30-05	M-3	4.9	30	1.14	2.0291	0.1479	0.1813	1.6545	1.493	9.4	21.876	0.151	3.157	0.457	1.360
M3-30-06			30	1.15	2.0648	0.1453	0.1792	1.6737	1.453	8.8	20.371	0.132	3.434	0.531	1.421
M3-30-07			30	1.20	2.2483	0.1334	0.1697	1.7676	1.608	7.5	17.726	0.098	2.881	0.523	1.399
M3-30-08			30	1.25	2.4395	0.1230	0.1607	1.8668	1.452	9.4	21.756	0.150	3.573	0.517	1.680
M3-30-09			30	1.35	2.8455	0.1054	0.1459	2.0568	1.257	6.7	15.115	0.074	4.173	0.847	2.264
M3-30-10			30	1.50	3.5129	0.0854	0.1283	2.3379	2.125	6.4	16.106	0.076	2.834	0.600	1.653
M4-20-01	M-4	10.3	20	0.85	1.1280	0.1773	0.1558	1.2834	1.047	5.9	28.609	0.261	9.583	1.051	1.078
M4-20-02			20	0.91	1.2929	0.1547	0.1872	1.0682	1.113	5.4	26.576	0.222	8.168	0.979	1.161
M4-20-03			20	0.95	1.4091	0.1419	0.1765	1.1330	1.037	4.7	22.742	0.165	7.536	1.038	1.359

M4-20-05			20	1.14	2.0291	0.0966	0.1399	1.4298	1.020	4.6	22.171	0.158	7.296	1.027	1.990
M4-20-06			20	1.15	2.0648	0.0969	0.1385	1.4438	1.475	4.1	21.631	0.137	7.579	1.197	1.400
M4-20-07			20	1.20	2.2483	0.0890	0.1312	1.5244	1.429	3.6	18.848	0.105	6.273	1.228	1.573
M4-20-08			20	1.25	2.4395	0.0820	0.1320	1.5152	1.632	4.5	24.309	0.169	5.924	0.852	1.495
M4-20-09			20	1.35	2.8455	0.0703	0.1143	1.7498	1.703	3.7	20.179	0.115	4.008	0.701	1.671
M4-20-10			20	1.50	3.5129	0.0569	0.1011	1.9785	1.462	2.1	11.056	0.036	4.160	1.282	2.403

M4-30-01			30	0.85	1.1280	0.2659	0.2823	1.0627	1.040	8.0	36.986	0.457	5.650	0.457	1.085
M4-30-02			30	0.91	1.2929	0.2320	0.2526	1.1876	1.289	6.6	31.427	0.321	4.662	0.457	1.003
M4-30-03			30	0.95	1.4091	0.2129	0.2363	1.2695	1.337	7.5	35.936	0.416	3.711	0.320	1.054
M4-30-04			30	1.10	1.8892	0.1588	0.1909	1.5713	1.420	7.3	35.364	0.399	5.419	0.481	1.330
M4-30-05	M-4	10.3	30	1.14	2.0291	0.1479	0.1813	1.6545	1.493	6.5	31.797	0.319	5.280	0.526	1.360
M4-30-06			30	1.15	2.0648	0.1453	0.1792	1.6737	1.453	5.5	26.762	0.227	3.711	0.437	1.421
M4-30-07			30	1.20	2.2483	0.1334	0.1697	1.7676	1.608	5.1	25.337	0.200	5.003	0.635	1.399
M4-30-08			30	1.25	2.4395	0.1230	0.1607	1.8668	1.452	4.5	21.893	0.152	4.108	0.591	1.680
M4-30-09			30	1.35	2.8455	0.1054	0.1459	2.0568	1.257	3.9	21.339	0.148	3.093	0.513	2.264
M4-30-10			30	1.50	3.5129	0.0854	0.1283	2.3379	2.125	4.9	25.920	0.196	2.917	0.385	1.653

Summary of Detailed Tests at 1.35s
Water depth, d = 20cm

D	draft	**Ht**	Transmitted wave height	**Ct**	Coefficient of transmission
L	Wavelength	**k**	Wave number	**x**	Displacement
Hi	Incident wave height	**F**	Wave force		

Test	Model	Cycle	D (cm)	L (m)	k	Hi (cm)	Ht (cm)	Ct	F (N)	x (cm)		x/L		H/d	
										Backward	Forward	Backward	Forward	Incident	Attenuated
DM1-20-01		1				2.8	1.1	0.3929	3.3977	-2.0	2.6	-0.0117	0.0153	0.1400	0.0550
DM1-20-02		2				2.8	1.0	0.3571	3.7897	-1.8	2.9	-0.0106	0.0170	0.1400	0.0500
DM1-20-03		3				3.0	1.0	0.3333	3.7897	-2.2	2.9	-0.0129	0.0170	0.1500	0.0500
DM1-20-04		4				3.0	1.0	0.3333	4.3124	-2.2	3.3	-0.0129	0.0194	0.1500	0.0500
DM1-20-05		5				3.0	1.1	0.3667	4.3124	-2.2	3.3	-0.0129	0.0194	0.1550	0.0550
DM1-20-06		6				3.0	1.0	0.3333	4.5739	-2.3	3.5	-0.0135	0.0206	0.1500	0.0500
DM1-20-07		7				3.0	1.0	0.3333	4.3124	-2.1	3.3	-0.0123	0.0194	0.1500	0.0500
DM1-20-08	M-1	8	6.0	1.7027	3.690	3.1	0.9	0.2903	3.6590	-2.1	2.8	-0.0123	0.0164	0.1550	0.0450
DM1-20-09		9				3.0	0.9	0.3000	3.7897	-2.2	2.9	-0.0129	0.0170	0.1500	0.0450
DM1-20-10		10				3.0	1.0	0.3333	4.0511	-2.2	3.1	-0.0129	0.0182	0.1500	0.0500
DM1-20-11		11				3.1	1.0	0.3226	5.2272	-2.0	4.0	-0.0117	0.0235	0.1550	0.0500
DM1-20-12		12				3.0	1.0	0.3333	4.9658	-2.0	3.8	-0.0117	0.0223	0.1500	0.0500
DM1-20-13		13				3.0	1.1	0.3667	4.4431	-2.1	3.4	-0.0123	0.0200	0.150.15	0.0550
DM1-20-14		14				3.0	1.1	0.3667	4.3124	-2.0	3.3	-0.0117	0.0194	0.1500	0.0550
DM1-20-15		15				3.1	1.0	0.3226	4.4431	-2.0	3.4	-0.0117	0.0200	0.1550	0.0500
DM2-20-01	M-2	1	6.5	1.7027	3.690	1.4	1.8	1.2857	3.1363	-1.1	2.4	-0.0065	0.0141	0.0700	0.0900
DM2-20-02		2				2.2	1.0	0.4545	2.3522	-0.7	1.8	-0.0041	0.0106	0.1100	0.0500
DM2-20-03		3				2.4	1.1	0.4583	2.8750	-0.8	2.2	-0.0047	0.0129	0.1200	0.0550
DM2-20-04		4				1.8	1.1	0.6111	3.2670	-1.1	2.5	-0.0650	0.0147	0.0900	0.0550
DM2-20-05		5				1.9	1.0	0.5263	2.3522	-0.3	1.8	-0.0018	0.0106	0.0950	0.0500
DM2-20-06		6				1.5	1.0	0.6667	2.2216	-0.6	1.7	-0.0035	0.0100	0.0750	0.0500
DM2-20-07		7				1.6	1.1	0.6875	1.9602	-0.3	1.5	-0.0018	0.0088	0.0800	0.0550
DM2-20-08		8				1.5	1.1	0.7333	2.3522	-0.9	1.8	-0.0053	0.0106	0.0750	0.0550
DM2-20-09		9				1.6	1.1	0.6875	2.3522	-0.5	1.8	-0.0029	0.0106	0.0800	0.0550
DM2-20-10		10				1.6	1.1	0.6875	2.6136	-0.6	2.0	-0.0035	0.0117	0.0800	0.0550
DM2-20-11		11				1.6	1.0	0.6250	2.7443	-1.0	2.1	-0.1006	0.0123	0.0800	0.0500
DM2-20-12		12				1.3	1.0	0.7692	2.4829	-0.5	1.9	-0.0029	0.0112	0.0650	0.0500

DM2-20-14		14				1.4	1.0	0.7143	2.3522	0.0	1.8	0.0000	0.0106	0.0700	0.0500
DM2-20-15		15				1.5	1.1	0.7333	2.4829	0.0	1.9	0.0000	0.1120	0.0750	0.0550

DM3-20-01		1				4.2	3.2	0.7619	1.9602	-2.2	1.5	-0.0129	0.0088	0.2100	0.1600
DM3-20-02		2				4.1	3.1	0.7561	1.9602	-2.5	1.5	-0.0147	0.0088	0.2050	0.1550
DM3-20-03		3				4.1	3.1	0.7561	1.9602	-2.4	1.5	-0.0141	0.0088	0.2050	0.1550
DM3-20-04		4				4.3	3.1	0.7209	1.9602	-2.0	1.5	-0.0117	0.0088	0.2150	0.1550
DM3-20-05		5				4.1	3.2	0.7805	1.5682	-2.3	1.2	-0.0135	0.0070	0.2050	0.1600
DM3-20-06		6				4.3	3.2	0.7442	1.8295	-2.2	1.4	-0.0129	0.0082	0.2150	0.1600
DM3-20-07		7				4.5	3.1	0.6889	1.9602	-2.2	1.5	-0.0129	0.0088	0.2250	0.1550
DM3-20-08	M-3	8	4.9	1.7027	3.690	4.4	3.3	0.7500	1.5682	-2.6	1.2	-0.0153	0.0070	0.2200	0.1650
DM3-20-09		9				4.3	3.1	0.7209	2.0909	-2.9	1.6	-0.0170	0.0094	0.2150	0.1550
DM3-20-10		10				4.1	3.1	0.7561	2.0909	-2.9	1.6	-0.0170	0.0094	0.2050	0.1550
DM3-20-11		11				4.2	3.1	0.7381	1.5682	-2.6	1.2	-0.0153	0.0070	0.2100	0.1550
DM3-20-12		12				4.2	3.2	0.7619	1.5682	-2.7	1.2	-0.0159	0.0070	0.2100	0.1600
DM3-20-13		13				4.1	3.1	0.7561	2.0909	-2.9	1.6	-0.0147	0.0094	0.2050	0.1550
DM3-20-14		14				4.0	3.3	0.8250	1.8295	-2.7	1.4	-0.0159	0.0082	0.2000	0.1650
DM3-20-15		15				4.0	3.2	0.8000	2.0909	-2.6	1.6	-0.0153	0.0094	0.2000	0.1600

DM4-20-01		1				2.2	1.9	0.8636	2.4829	-2.2	1.9	-0.0129	0.0112	0.1100	0.0950
DM4-20-02		2				2.5	1.8	0.7200	3.2670	-2.5	2.5	-0.0147	0.0147	0.1250	0.0900
DM4-20-03		3				3.0	1.8	0.6000	3.5284	-1.9	2.7	-0.0112	0.0159	0.1500	0.0900
DM4-20-04		4				3.1	1.9	0.6129	3.3977	-2.9	2.6	-0.0170	0.0153	0.1550	0.0950
DM4-20-05		5				2.9	1.8	0.6207	3.3977	-2.2	2.6	-0.0129	0.0153	0.1450	0.0900
DM4-20-06		6				3.0	1.8	0.6000	2.8750	-2.2	2.2	-0.0129	0.0129	0.1500	0.0900
DM4-20-07		7				2.9	1.7	0.5862	3.3977	-1.7	2.6	-0.0100	0.0153	0.1450	0.0850
DM4-20-08	M-4	8	10.3	1.7027	3.690	3.1	1.7	0.5484	2.7443	-1.8	2.1	-0.0106	0.0123	0.1550	0.0850
DM4-20-09		9				3.0	1.8	0.6000	2.3522	-1.5	1.8	-0.0088	0.0106	0.1500	0.0900
DM4-20-10		10				2.7	1.7	0.6296	2.4829	-1.4	1.9	-0.0082	0.0110	0.1350	0.0850
DM4-20-11		11				2.7	1.9	0.7037	1.4376	-1.2	1.1	-0.0070	0.0065	0.1350	0.0950
DM4-20-12		12				2.9	2.1	0.7241	2.2216	-1.2	1.7	-0.0070	0.0100	0.1450	0.1050
DM4-20-13		13				2.4	2.0	0.8333	1.6988	-1.3	1.3	-0.0076	0.0076	0.1200	0.1000
DM4-20-14		14				2.3	2.0	0.8696	1.6988	-1.1	1.3	-0.0065	0.0076	0.1150	0.1000
DM4-20-15		15				2.5	2.0	0.8000	1.9602	-1.2	1.5	-0.0070	0.0088	0.1250	0.1000

Water depth, d = 30cm

L	Wavelength	k	Wave number	x	Displacement
Hi	Incident wave height	F	Wave force		

Test	Model	Cycle	D (cm)	L (m)	k	Hi (cm)	Ht (cm)	Ct	F (N)	x (cm)		x/L		H/d	
										Backward	Forward	Backward	Forward	Incident	Attnuated
DM1-30-01		1				10.1	4.3	0.426	7.972	-4.8	6.1	-0.038	0.049	0.080	0.034
DM1-30-02		2				8.9	6.3	0.708	4.835	-5.8	3.7	-0.046	0.029	0.071	0.050
DM1-30-03		3				10.8	5.6	0.519	8.886	-3.2	6.8	-0.026	0.054	0.086	0.045
DM1-30-04		4				9.1	6.2	0.681	1.960	-5.8	1.5	-0.046	0.012	0.072	0.049
DM1-30-05		5				11.2	5.8	0.518	13.199	-4.9	10.1	-0.039	0.080	0.089	0.046
DM1-30-06		6				9.3	6.0	0.645	1.438	-5.6	1.1	-0.045	0.009	0.074	0.048
DM1-30-07		7				11.6	6.1	0.526	12.545	-4.2	9.6	-0.033	0.076	0.092	0.049
DM1-30-08	M-1	8	6.0	1.257	4.998	11.4	6.7	0.588	0.392	-3.9	0.3	-0.031	0.002	0.091	0.053
DM1-30-09		9				10.6	5.8	0.547	9.540	-5.7	7.3	3.045	0.058	0.084	0.046
DM1-30-10		10				10.3	6.3	0.612	0.653	-4.0	0.5	-0.032	0.004	0.082	0.050
DM1-30-11		11				9.7	6.1	0.629	10.324	-4.0	7.9	-0.032	0.063	0.077	0.049
DM1-30-12		12				10.1	5.5	0.545	0.000	-4.9	0.0	-0.039	0.000	0.088	0.044
DM1-30-13		13				9.2	5.8	0.630	14.113	-2.4	10.8	-0.019	0.086	0.073	0.046
DM1-30-14		14				10.2	5.8	0.569	1.438	-6.6	1.1	-0.053	0.009	0.081	0.046
DM1-30-15		15				8.9	5.9	0.663	17.511	-2.1	13.4	-0.017	0.107	0.071	0.047
DM2-30-01	M-2	1	6.5	1.257	4.998	5.8	4.1	0.707	6.534	-3.6	5.0	-0.029	0.040	0.193	0.137
DM2-30-02		2				6.0	3.8	0.633	7.187	-5.1	5.5	-0.041	0.044	0.200	0.127
DM2-30-03		3				6.1	4.1	0.672	5.489	-2.8	4.2	-0.022	0.033	0.203	0.137
DM2-30-04		4				7.1	2.4	0.338	7.187	-3.7	5.5	-0.029	0.044	0.237	0.080
DM2-30-05		5				6.3	2.2	0.349	8.886	-3.8	6.8	-0.030	0.054	0.210	0.073
DM2-30-06		6				6.3	3.1	0.492	6.665	-4.1	5.1	-0.033	0.041	0.210	0.103
DM2-30-07		7				7.8	4.3	0.551	6.273	-4.2	4.8	-0.033	0.038	0.260	0.143
DM2-30-08		8				7.2	5.8	0.806	6.795	-2.2	5.2	-0.018	0.041	0.240	0.193
DM2-30-09		9				7.1	6.0	0.845	3.398	-3.2	2.6	-0.026	0.021	0.234	0.200
DM2-30-10		10				7.2	6.0	0.833	9.932	-2.6	7.6	-0.021	0.061	0.240	0.200
DM2-30-11		11				6.1	6.1	1.000	12.023	-2.9	9.2	-0.023	0.073	0.203	0.203
DM2-30-12		12				6.4	4.2	0.656	0.392	-3.6	0.3	-0.029	0.002	0.213	0.140
DM2-30-13		13				5.6	3.7	0.661	12.545	-2.7	9.6	-0.022	0.076	0.187	0.123

[illegible]		15				6.3	4.1	0.651	11.369	-4.8	8.7	-0.038	0.069	0.210	0.137
DM3-30-01		1				9.3	5.6	0.602	7.972	-5.2	6.1	-0.041	0.049	0.310	0.187
DM3-30-02		2				9.3	5.9	0.634	8.494	-5.0	6.5	-0.040	0.052	0.310	0.197
DM3-30-03		3				9.1	7.3	0.802	5.881	-5.2	4.5	-0.041	0.036	0.303	0.243
DM3-30-04		4				9.0	4.8	0.533	8.102	-5.0	6.2	-0.040	0.049	0.300	0.160
DM3-30-05		5				9.5	6.3	0.663	7.710	-6.5	5.9	-0.052	0.047	0.317	0.210
DM3-30-06		6				10.0	5.1	0.510	4.051	-4.8	3.1	-0.038	0.025	0.333	0.170
DM3-30-07		7				8.2	5.1	0.622	7.972	-5.6	6.1	-0.045	0.049	0.273	0.170
DM3-30-08	M-3	8	4.9	1.257	4.998	12.0	4.3	0.358	4.574	-4.3	3.5	-0.034	0.028	0.400	0.143
DM3-30-09		9				8.3	5.3	0.639	7.318	-5.0	5.6	-0.040	0.045	0.277	0.177
DM3-30-10		10				12.4	4.2	0.339	3.528	-5.7	2.7	-0.045	0.022	0.413	0.140
DM3-30-11		11				9.6	5.2	0.542	10.585	-5.6	8.1	-0.045	0.064	0.320	0.173
DM3-30-12		12				13.0	5.7	0.438	5.619	-6.8	4.3	-0.054	0.034	0.433	0.190
DM3-30-13		13				9.4	6.1	0.649	9.148	-7.0	7.0	-0.056	0.056	0.313	0.203
DM3-30-14		14				7.5	6.1	0.813	8.886	-5.6	6.8	-0.045	0.054	0.250	0.203
DM3-30-15		15				9.8	6.2	0.633	10.193	-6.0	7.8	-0.048	0.062	0.327	0.207
DM4-30-01		1				6.2	3.8	0.613	9.409	-2.3	7.2	-0.018	0.057	0.207	0.127
DM4-30-02		2				5.8	3.8	0.655	9.540	-2.5	7.3	-0.020	0.058	0.193	0.127
DM4-30-03		3				6.3	3.9	0.619	10.585	-1.7	8.1	-0.014	0.064	0.210	0.130
DM4-30-04		4				6.2	3.9	0.629	8.364	-3.1	6.4	-0.025	0.051	0.207	0.130
DM4-30-05		5				6.0	4.0	0.667	10.454	-2.4	8.0	-0.019	0.064	0.200	0.133
DM4-30-06		6				6.0	4.1	0.683	8.494	-2.6	6.5	-0.021	0.052	0.200	0.137
DM4-30-07		7				5.9	4.2	0.712	9.148	-2.7	7.0	-0.022	0.056	0.197	0.140
DM4-30-08	M-4	8	10.3	1.257	4.998	5.8	4.2	0.724	10.324	-1.8	7.9	-0.014	0.063	0.193	0.140
DM4-30-09		9				5.9	4.2	0.712	8.756	-2.0	6.7	-0.016	0.053	0.197	0.140
DM4-30-10		10				6.0	4.0	0.667	9.409	-2.3	7.2	-0.018	0.057	0.200	0.133
DM4-30-11		11				6.0	4.0	0.667	9.801	-2.4	7.5	-0.019	0.060	0.200	0.133
DM4-30-12		12				6.2	4.0	0.645	8.364	-2.4	6.4	-0.019	0.051	0.207	0.133
DM4-30-13		13				6.2	4.0	0.645	9.540	-2.6	7.3	-0.021	0.058	0.207	0.133
DM4-30-14		14				6.2	4.0	0.645	9.148	-2.9	7.0	-0.023	0.056	0.207	0.133
DM4-30-15		15				6.2	4.0	0.645	10.193	-1.9	7.8	-0.015	0.062	0.207	0.133

APPENDIX B

Tabulated Data and Calculations (Chapter 4)

B: 1 Calculations of Stability

B: 2 Calculations of Wave Length

B: 3 Tabulated Data for Wave Heights

B: 4 Tabulated Data for Theoretical Wave Forces Calculations

B: 5 Tabulated Data for Calibration (Force Vs. Displacement) Graph

B: 6 Tabulated Data for Experimental Wave Forces Calculations

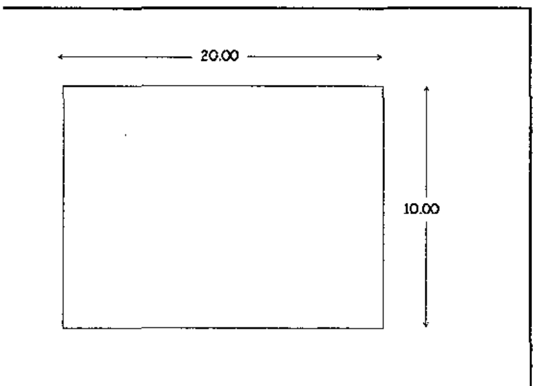
B: 7 Tabulated Data for Standard Statistic of Detailed Study of Wave Forces at T = 1.35s

B: 8 Distribution of Energy for Detailed Study

3: 1 CALCULATIONS OF STABILITY

The calculations below are based on preliminary research and there were assumptions nade in the density of wood. Therefore the weight of the wood would vary with the ıctual weight of the model.

Л-1 Model



he weight of the model is 0.92kg.

Vith **draft of 60mm**, density of water = 1000 kg/m^3 = 0.001g/mm^3

$$W + 920) = 60 \times 200 \times 300 \times 0.001$$
$$V = 2680g = 2.68kg$$

xtra weight needed is **2.68kg**.

tability of the block:

enter of gravity, **G is 5.0cm** from the bottom of the block.

or a regularly shaped body, the center of buoyancy, B is the part of half of the height nmersed body.

$$3 = \frac{6}{2} = 3cm$$

enter of buoyancy, **B is 3.0cm** from the bottom of the block.

)istance **GB = 5 – 3 = 2cm**

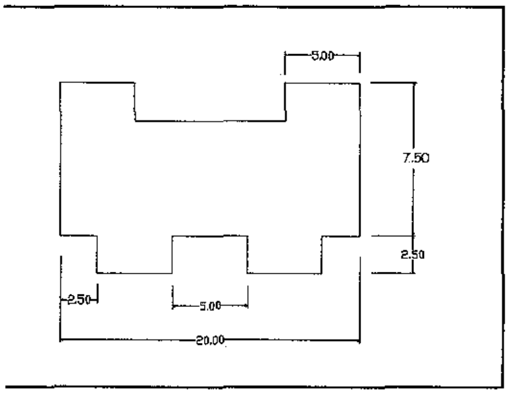
$$MB = \frac{I}{V_s} = \frac{1/12bh^3}{V_s} = \frac{1/12 \times 30 \times 20^3}{(20 \times 10 \times 30)} = 3.333cm$$

'herefore, **MB = 3.333cm**

/letacentric height, MG = MB – GB = 3.333 – 2.0 = 1.333cm

/IG > 0, therefore the block is **stable.**

/I-2 Model



xtra mass needed for desired draft:

he weight of the model is 1.1kg.

/ith **draft of 65mm,** density of water = 1000 kg/m^3 = 0.001g/mm^3

$$W + 1100) = [(50 \times 25 \times 300) \times 2 + (40 \times 200 \times 300)] \times 0.001$$
$$W = 20500g = 2.05kg$$

xtra weight needed is **2.05kg**.

tability of the block:

$$\bar{y} = \frac{[(5 \times 2.5 \times 11.25) \times 2] + [(5 \times 2.5 \times 1.25) \times 2] + (7.5 \times 20 \times 6.25)}{[(5 \times 2.5) \times 4] + (7.5 \times 20)} = 6.25cm$$

Center of gravity, **G is 6.25cm** from the bottom of the block.
For a regularly shaped body, the center of buoyancy, B is the part of half of the height mmersed body.

$$B = \frac{[(5\times2.5\times1.25)\times2]+(20\times4\times4.5)}{[(5\times2.5)\times2]+(20\times4)} = 3.726cm$$

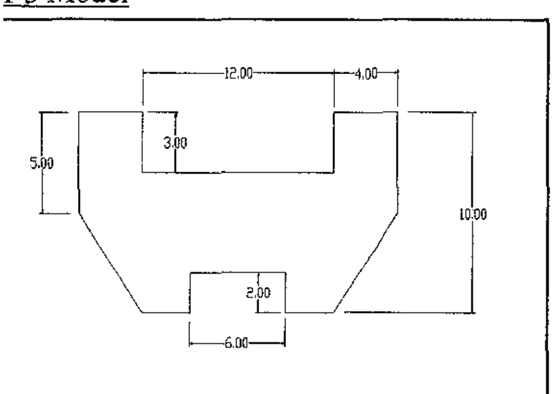
Center of buoyancy, **B is 3.726cm** from the bottom of the block.

Distance **GB = 6.25 – 3.726 = 2.524cm**

$$MB = \frac{I}{V_s} = \frac{1/12bh^3}{V_s} = \frac{1/12\times30\times20^3}{(4\times20\times30)+(2.5\times5\times30)\times2} = 6.34921cm$$

Therefore, **MB = 6.35cm**
Metacentric height, MG = MB – GB = 6.35 – 2.524 = 3.826cm
MG > 0, therefore the block is **stable**.

1-3 Model



xtra mass needed for desired draft:
he weight of the model is 0.91kg.

With **draft of 49mm**, density of water = 1000 kg/m^3 = 0.001g/mm^3

$$(V + 910) = [(20\times30\times300)\times2 + (29\times120\times300) + (1/2\times39.2\times49\times300]\times0.001$$
$$V = 782.12g = 0.78212kg$$

Extra weight needed is **0.78kg**

Stability of the block:

$$G = \frac{[(3\times4\times8.5)\times2]+(2\times20\times6)+(3\times6\times3.5)+[(3\times5\times2.5)\times2]+[(1/2\times4\times5\times10/3)\times2]}{[(3\times4)\times2]+(2\times20)+(3\times6)+[(3\times5)\times2]+[(1/2\times4\times5)\times2]} = 4.63cm$$

Center of gravity, **G is 4.63cm** from the bottom of the block.

For a regularly shaped body, the center of buoyancy, B is the part of half of the height immersed body.

$$B = \frac{[(1/2\times5\times4\times5/3)\times2]+[(3\times5\times2.5)\times2]+(6\times3\times3.5)+(1\times20\times5.5)}{[(1/2\times4\times5)\times2]+[(3\times5)\times2]+(6\times3)+(1\times20)} = 3.01cm$$

Center of buoyancy, **B is 2.5cm** from the bottom of the block.

Distance **GB = 4.63 – 3.01 = 1.62cm**

$$MB = \frac{I}{V_s} = \frac{1/12bh^3}{V_s} = \frac{1/12\times30\times20^3}{[(1/2\times(7+3)\times5\times30)\times2]+(3\times6\times30)} = 9.803922cm$$

Therefore, **MB = 9.804cm**

Metacentric height, MG = MB – GB = 9.804 – 1.62 = 8.184cm

MG > 0, therefore the block is **stable.**

B: 2 CALCULATIONS OF WAVE LENGTH

$L_0 = \dfrac{gT^2}{2\pi}$ Where T is the wave period.

d/L is obtained from Table C-1 of the Shore Protection Manual.

For water depth of 0.2m:

T = 1.0s

$L_0 = \dfrac{9.81\times1.0^2}{2\pi} = 1.561$

$d/L_0 = 0.2/1.561 = 0.1281$

$d/L = 0.165$

$L = d/0.165 = 0.2/0.165 = 1.212m$

T = 1.1s

$L_0 = \dfrac{9.81\times1.1^2}{2\pi} = 1.889$

$d/L_0 = 0.2/1.889 = 0.1077$

$d/L = 0.1476$

$L = d/0.1476 = 0.2/0.1479 = 1.3548m$

T = 1.2s

$L_0 = \dfrac{9.81\times1.2^2}{2\pi} = 2.248$

$d/L_0 = 0.2/2.248 = 0.089$

$d/L = 0.1313$

$L = d/0.1313 = 0.2/0.1313 = 1.5232m$

T = 1.3s

$L_0 = \dfrac{9.81\times1.3^2}{2\pi} = 2.639$

$d/L_0 = 0.2/2.639 = 0.0786$

$d/L = 0.122$

$L = d/0.122 = 0.2/0.122 = 1.6399m$

= 1.4s

$L_0 = \dfrac{9.81\times1.4^2}{2\pi} = 3.060$

$d/L_0 = 0.2/3.060 = 0.0658$

$d/L = 0.101$

$L = d/0.101 = 0.2/0.101 = 1.8187m$

t = 1.5s

$L_0 = \dfrac{9.81\times1.5^2}{2\pi} = 3.513$

$d/L_0 = 0.2/3.513 = 0.0598$

$d/L = 0.1041$

$L = d/0.1041 = 0.2/0.1041 = 1.9204m$

= 1.6s

$L_0 = \dfrac{9.81\times1.6^2}{2\pi} = 3.997$

$d/L_0 = 0.2/3.997 = 0.0514$

$d/L = 0.956$

$L = d/0.956 = 0.2/0.956 = 2.0916m$

T = 1.7s

$L_0 = \dfrac{9.81\times1.7^2}{2\pi} = 4.512$

$d/L_0 = 0.2/4.512 = 0.0461$

$d/L = 0.0901$

$L = d/0.0901 = 0.2/0.0901 = 2.2209m$

T = 1.8s

$$L_0 = \frac{9.81\times1.8^2}{2\pi} = 5.059$$
$d/L_0 = 0.2/5.059 = 0.0388$
$d/L = 0.0819$
$L = d/0.0388 = 0.2/0.0388 = 2.4426m$

T = 2.0s

$$L_0 = \frac{9.81\times2.0^2}{2\pi} = 6.245$$
$d/L_0 = 0.2/6.245 = 0.032$
$d/L = 0.0739$
$L = d/0.0739 = 0.2/0.0739 = 2.7066m$

For water depth of 0.3m:

T = 1.0s

$$L_0 = \frac{9.81\times1.0^2}{2\pi} = 1.561$$
$d/L_0 = 0.3/1.561 = 0.1922$
$d/L = 0.2186$
$L = d/0.2186 = 0.2/0.2186 = 1.3725m$

T = 1.1s

$$L_0 = \frac{9.81\times1.1^2}{2\pi} = 1.889$$
$d/L_0 = 0.3/1.889 = 0.1615$
$d/L = 0.1907$
$L = d/0.1907 = 0.2/0.1907 = 1.5732m$

T = 1.2s

$$L_0 = \frac{9.81\times1.2^2}{2\pi} = 2.248$$
$d/L_0 = 0.3/2.248 = 0.1335$
$d/L = 0.1695$
$L = d/0.1695 = 0.2/0.1695 = 1.7699m$

T = 1.3s

$$L_0 = \frac{9.81\times1.3^2}{2\pi} = 2.639$$
$d/L_0 = 0.3/2.639 = 0.1179$
$d/L = 0.1528$
$L = d/0.1528 = 0.2/0.1528 = 1.9632m$

T = 1.4s

$$L_0 = \frac{9.81\times1.4^2}{2\pi} = 3.060$$
$d/L_0 = 0.3/3.060 = 0.0987$
$d/L = 0.1393$
$L = d/0.1393 = 0.2/0.1393 = 2.1536m$

t = 1.5s

$$L_0 = \frac{9.81\times1.5^2}{2\pi} = 3.513$$
$d/L_0 = 0.3/3.513 = 0.0897$
$d/L = 0.1281$
$L = d/0.1281 = 0.2/0.1281 = 2.3416m$

T = 1.6s

$$L_0 = \frac{9.81\times1.6^2}{2\pi} = 3.997$$
$d/L_0 = 0.3/3.997 = 0.0771$
$d/L = 0.1187$
$L = d/0.1187 = 0.2/0.1187 = 2.5275m$

T = 1.7s

$$L_0 = \frac{9.81\times1.7^2}{2\pi} = 4.512$$
$d/L_0 = 0.3/4.512 = 0.0692$
$d/L = 0.1106$
$L = d/0.1106 = 0.2/0.1106 = 2.7118m$

T = 1.8s

$$L_0 = \frac{9.81 \times 1.8^2}{2\pi} = 5.059$$

$d/L_0 = 0.3/5.059 = 0.0581$

$d/L = 0.1036$

$L = d/0.1036 = 0.2/0.1036 = 2.8947m$

T = 2.0s

$$L_0 = \frac{9.81 \times 2.0^2}{2\pi} = 6.245$$

$d/L_0 = 0.3/6.245 = 0.048$

$d/L = 0.0921$

$L = d/0.0921 = 0.2/0.0921 = 3.2571m$

B: 3 TABULATED DATA FOR WAVE HEIGHTS

Table B-1 Determination of Incident Wave Height at Depth of 20cm for M-1

Wave Period, T(s)	1	2	3	Average	H max
0.85	7.50	7.60	7.80	7.63	7.80
0.91	7.00	7.20	7.10	7.10	7.20
0.95	6.80	6.50	6.60	6.63	6.80
1.10	6.20	6.40	6.00	6.20	6.40
1.14	5.70	5.80	6.00	5.83	6.00
1.15	5.40	5.30	5.50	5.40	5.50
1.20	5.50	5.30	5.00	5.27	5.50
1.25	4.20	4.20	4.30	4.23	4.30
1.35	3.90	4.10	3.80	3.93	4.10
1.40	3.80	3.60	4.00	3.80	4.00
1.50	3.00	3.10	3.10	3.07	3.10

Table B-2 Determination of Incident Wave Height at Depth of 30cm for M-1

Wave Period, T(s)	1	2	3	Average	H max
0.85	10.20	10.40	10.00	10.20	10.40
0.91	9.10	9.20	9.30	9.20	9.30
0.95	8.10	9.00	8.30	8.47	9.00
1.10	8.10	7.80	7.80	7.90	8.10
1.14	7.30	7.10	7.20	7.20	7.30
1.15	4.50	5.00	4.00	4.50	5.00
1.20	6.30	6.00	5.90	6.07	6.30
1.25	4.10	4.30	4.30	4.23	4.30
1.35	5.80	5.20	5.80	5.60	5.80
1.40	7.00	6.50	6.20	6.57	7.00
1.50	2.10	2.00	1.70	1.93	2.10

Table B-3 Determination of Incident Wave Height at Depth of 20cm for M-2

Wave Period, T(s)	1	2	3	Average	H max
0.85	6.80	7.10	7.10	7.00	7.10
0.91	6.20	6.50	6.00	6.23	6.50
0.95	5.90	6.00	6.10	6.00	6.10
1.10	5.50	5.00	5.20	5.23	5.50
1.14	3.50	3.70	4.20	3.80	4.20
1.15	2.90	2.50	3.30	2.90	3.30
1.20	7.50	7.50	7.20	7.40	7.50
1.25	4.80	4.80	4.50	4.70	4.80
1.35	4.50	4.00	4.80	4.43	4.80
1.40	4.50	3.70	5.50	4.57	5.50
1.50	2.50	2.70	3.10	2.77	3.10

Table B-4 Determination of Incident Wave Height at Depth of 30cm for M-2

Wave Period, T(s)	1	2	3	Average	H max
0.85	9.30	8.70	9.50	9.17	9.50
0.91	10.00	9.60	9.30	9.63	10.00
0.95	7.20	7.70	8.20	7.70	8.20
1.10	7.20	7.00	7.50	7.23	7.50
1.14	6.30	6.70	7.50	6.83	7.50
1.15	7.20	8.00	7.30	7.50	8.00
1.20	6.60	6.20	6.30	6.37	6.60
1.25	5.80	5.90	6.00	5.90	6.00
1.35	6.80	7.20	6.20	6.73	7.20
1.40	6.20	5.90	5.90	6.00	6.20
1.50	4.90	4.80	4.00	4.57	4.90

Table B-5 Determination of Incident Wave Height at Depth of 20cm for M-3

Wave Period, T(s)	1	2	3	Average	H max
0.85	6.40	7.20	7.10	6.90	7.20
0.91	7.50	7.00	7.20	7.23	7.50
0.95	6.40	6.30	6.60	6.43	6.60
1.10	7.00	6.90	6.50	6.80	7.00
1.14	5.50	5.70	5.70	5.63	5.70
1.15	4.50	4.50	4.50	4.50	4.50
1.20	6.50	6.70	6.40	6.53	6.70
1.25	5.00	5.80	5.40	5.40	5.80
1.35	4.80	4.70	4.70	4.73	4.80
1.40	4.20	4.50	4.30	4.33	4.50
1.50	3.90	4.20	4.30	4.13	4.30

Table B-6 Determination of Incident Wave Height at Depth of 30cm for M-3

Wave Period, T(s)	1	2	3	Average	H max
0.85	9.00	8.80	8.40	8.73	9.00
0.91	8.90	9.40	9.30	9.20	9.40
0.95	10.20	10.00	10.00	10.07	10.20
1.10	10.50	10.40	10.50	10.47	10.50
1.14	9.00	9.40	9.30	9.23	9.40
1.15	8.70	8.60	8.80	8.70	8.80
1.20	7.00	7.50	7.20	7.23	7.50
1.25	9.30	9.40	9.00	9.23	9.40
1.35	6.70	6.50	6.50	6.57	6.70
1.40	6.20	6.60	6.40	6.40	6.60
1.50	6.40	6.10	6.40	6.30	6.40

Table B-7 Determination of Incident Wave Height at Depth of 20cm for M-4

Wave Period, T(s)	1	2	3	Average	H max
0.85	5.6	5.7	5.9	5.7	5.9
0.91	5.1	5.3	5.4	5.3	5.4
0.95	4.6	4.4	4.7	4.6	4.7
1.10	4.4	4.2	4.4	4.3	4.4
1.14	4.2	4.6	4.2	4.3	4.6
1.15	3.8	4.1	3.9	3.9	4.1
1.20	3.6	3.4	3.2	3.4	3.6
1.25	4.4	4.5	4.5	4.5	4.5
1.35	3.2	3.6	3.7	3.5	3.7
1.40	2.4	2.3	2.0	2.2	2.4
1.50	2.0	1.9	2.1	2.0	2.1

Table B-8 Determination of Incident Wave Height at Depth of 30cm for M-4

Wave Period, T(s)	1	2	3	Average	H max
0.85	7.8	7.7	8.0	7.8	8.0
0.91	6.5	6.4	6.6	6.5	6.6
0.95	7.5	6.9	7.3	7.2	7.5
1.10	7.3	7.2	6.8	7.1	7.3
1.14	6.3	6.5	6.1	6.3	6.5
1.15	5.5	5.4	5.5	5.5	5.5
1.20	4.9	4.9	5.1	5.0	5.1
1.25	4.2	4.3	4.5	4.3	4.5
1.35	3.8	3.8	3.9	3.8	3.9
1.40	3.4	3.5	3.7	3.5	3.7
1.50	3.9	3.8	4.9	4.2	4.9

At water depth, d = 0.2m

Wave Period	L (m)	k	α_1	α_2	P_1 (kN/m^2)	F_1 (N)	P_2 (kN/m^2)	F_2 (N)	F_{ave} (N)	$F/(0.5\gamma AH_{max})$	$gT^2/2\pi L$
0.85	1.047	6.004	0.696	0.866	1.295	23.312	1.121	20.178	21.745	0.1526521	1.077923
0.91	1.113	5.644	0.714	0.876	1.208	21.745	1.058	19.042	20.393	0.1321495	1.161341
0.95	1.037	6.059	0.694	0.864	1.127	20.293	0.974	17.535	18.914	0.1157536	1.358806
1.10	1.249	5.032	0.750	0.893	1.096	19.734	0.980	17.632	18.683	0.1076133	1.513043
1.14	1.020	6.162	0.689	0.861	0.992	17.858	0.854	15.380	16.619	0.0897418	1.989878
1.15	1.475	4.259	0.806	0.917	0.972	17.500	0.891	16.040	16.770	0.0830096	1.399602
1.20	1.429	4.396	0.795	0.912	0.966	17.396	0.882	15.873	16.634	0.0823393	1.573108
1.25	1.632	3.851	0.839	0.929	0.774	13.937	0.719	12.944	13.440	0.0520141	1.495095
1.35	1.703	3.690	0.853	0.933	0.744	13.389	0.694	12.499	12.944	0.0477629	1.671162
1.50	1.462	4.297	0.803	0.915	0.547	9.847	0.501	9.014	9.431	0.0263113	2.402508

At water depth, d = 0.3m

Wave Period	L (m)	k	α_1	α_2	P_1 (kN/m^2)	F_1 (N)	P_2 (kN/m^2)	F_2 (N)	F_{ave} (N)	$F/(0.5\gamma AH_{max})$	$gT^2/2\pi L$
0.85	1.040	6.044	0.619	0.864	1.648	29.662	1.423	25.614	27.638	0.2586924	1.085182
0.91	1.289	4.873	0.650	0.888	1.502	27.032	1.334	24.005	25.518	0.2135894	1.002731
0.95	1.337	4.698	0.657	0.892	1.460	26.277	1.302	23.445	24.861	0.2013724	1.053598
1.10	1.420	4.424	0.670	0.899	1.324	23.840	1.191	21.435	22.638	0.1650292	1.330318
1.14	1.493	4.210	0.683	0.905	1.202	21.644	1.088	19.582	20.613	0.135429	1.359517
1.15	1.453	4.324	0.676	0.902	0.820	14.765	0.740	13.314	14.039	0.0631777	1.421082
1.20	1.608	3.908	0.703	0.913	1.050	18.904	0.959	17.259	18.081	0.1025208	1.398536
1.25	1.452	4.328	0.676	0.902	0.705	12.697	0.636	11.448	12.072	0.0467189	1.680244
1.35	1.257	4.998	0.645	0.885	0.934	16.810	0.827	14.878	15.844	0.0827077	2.263533
1.50	2.125	2.956	0.792	0.941	0.368	6.631	0.347	6.239	6.435	0.0121625	1.652841

Wave Period	L (m)	k	α_1	α_2	P_1 (kN/m^2)	F_1 (N)	P_2 (kN/m^2)	F_2 (N)	F_{ave} (N)	$F/(0.5\gamma A H_{max})$	$gT^2/2\pi L$
0.85	1.047	6.005	0.696	0.854	1.179	22.988	1.007	19.640	21.314	0.1475449	1.077923
0.91	1.113	5.644	0.714	0.865	1.091	21.266	0.944	18.402	19.834	0.1256987	1.161341
0.95	1.037	6.060	0.694	0.853	1.011	19.721	0.862	16.816	18.269	0.1086523	1.358806
1.10	1.249	5.033	0.750	0.885	0.942	18.372	0.833	16.251	17.311	0.0928323	1.513043
1.14	1.020	6.163	0.689	0.850	0.694	13.542	0.590	11.506	12.524	0.0512854	1.989878
1.15	1.475	4.259	0.806	0.910	0.583	11.374	0.531	10.346	10.860	0.0349432	1.399602
1.20	1.429	4.397	0.795	0.905	1.318	25.698	1.193	23.260	24.479	0.1790002	1.573108
1.25	1.632	3.851	0.839	0.923	0.864	16.854	0.798	15.552	16.203	0.0758295	1.495095
1.35	1.703	3.691	0.853	0.928	0.871	16.981	0.808	15.757	16.369	0.0766071	1.671162
1.50	1.462	4.298	0.803	0.908	0.547	10.667	0.497	9.690	10.179	0.0307646	2.402508

At water depth, d = 0.3m

Wave Period	L (m)	k	α_1	α_2	P_1 (kN/m^2)	F_1 (N)	P_2 (kN/m^2)	F_2 (N)	F_{ave} (N)	$F/(0.5\gamma A H_{max})$	$gT^2/2\pi L$
0.85	1.040	6.045	0.619	0.852	1.505	29.352	1.283	25.013	27.183	0.2517813	1.085182
0.91	1.289	4.874	0.650	0.879	1.615	31.489	1.419	27.667	29.578	0.2883867	1.002731
0.95	1.337	4.699	0.657	0.883	1.330	25.936	1.175	22.907	24.421	0.1952493	1.053598
1.10	1.420	4.425	0.670	0.891	1.226	23.914	1.092	21.299	22.606	0.1653099	1.330318
1.14	1.493	4.210	0.683	0.897	1.235	24.090	1.108	21.603	22.847	0.1670655	1.359517
1.15	1.453	4.325	0.676	0.894	1.312	25.592	1.173	22.867	24.230	0.1889912	1.421082
1.20	1.608	3.909	0.703	0.906	1.100	21.454	0.996	19.431	20.442	0.1315471	1.398536
1.25	1.452	4.328	0.676	0.893	0.984	19.192	0.879	17.147	18.169	0.1062905	1.680244
1.35	1.257	4.999	0.645	0.875	1.159	22.607	1.015	19.792	21.199	0.1488185	2.263533
1.50	2.125	2.957	0.792	0.936	0.860	16.762	0.805	15.688	16.225	0.0775144	1.652841

Wave Period	L (m)	k	α_1	α_2	P_1 (kN/m^2)	F_1 (N)	P_2 (kN/m^2)	F_2 (N)	F_{ave} (N)	$F/(0.5\gamma AH_{max})$	$gT^2/2\pi L$
0.85	1.047	6.004	0.696	0.890	1.195	17.574	1.064	15.645	16.609	0.0878957	1.077923
0.91	1.113	5.644	0.714	0.898	1.258	18.498	1.131	16.620	17.559	0.0967961	1.161341
0.95	1.037	6.059	0.694	0.889	1.094	16.085	0.973	14.300	15.193	0.0736991	1.358806
1.10	1.249	5.032	0.750	0.913	1.199	17.627	1.095	16.093	16.860	0.0867459	1.513043
1.14	1.020	6.162	0.689	0.887	0.942	13.855	0.836	12.285	13.070	0.0547554	1.989878
1.15	1.475	4.259	0.806	0.932	0.795	11.693	0.741	10.896	11.295	0.0373568	1.399602
1.20	1.429	4.396	0.795	0.929	1.177	17.306	1.093	16.069	16.687	0.0821771	1.573108
1.25	1.632	3.851	0.839	0.942	1.044	15.352	0.984	14.459	14.905	0.0635421	1.495095
1.35	1.703	3.690	0.853	0.946	0.871	12.801	0.824	12.106	12.454	0.0439365	1.671162
1.50	1.462	4.297	0.803	0.931	0.759	11.155	0.706	10.384	10.769	0.0340366	2.402508

At water depth, d = 0.3m

Wave Period	L (m)	k	α_1	α_2	P_1 (kN/m^2)	F_1 (N)	P_2 (kN/m^2)	F_2 (N)	F_{ave} (N)	$F/(0.5\gamma AH_{max})$	$gT^2/2\pi L$
0.85	1.040	6.044	0.619	0.889	1.426	20.963	1.267	18.627	19.795	0.1309431	1.085182
0.91	1.289	4.873	0.650	0.909	1.518	22.314	1.379	20.273	21.293	0.1471151	1.002731
0.95	1.337	4.698	0.657	0.912	1.654	24.321	1.509	22.180	23.250	0.1743079	1.053598
1.10	1.420	4.424	0.670	0.918	1.717	25.239	1.575	23.159	24.199	0.1867535	1.330318
1.14	1.493	4.210	0.683	0.922	1.548	22.761	1.428	20.990	21.876	0.1511383	1.359517
1.15	1.453	4.324	0.676	0.920	1.444	21.222	1.328	19.519	20.371	0.1317572	1.421082
1.20	1.608	3.908	0.703	0.929	1.250	18.378	1.161	17.073	17.726	0.0977123	1.398536
1.25	1.452	4.328	0.676	0.920	1.542	22.667	1.418	20.846	21.756	0.1503144	1.680244
1.35	1.257	4.998	0.645	0.906	1.079	15.859	0.978	14.370	15.115	0.0744316	2.263533
1.50	2.125	2.956	0.792	0.952	1.123	16.504	1.069	15.707	16.106	0.0757621	1.652841

At water depth, d = 0.2m

Wave Period	L (m)	k	α_1	α_2	P_1 (kN/m^2)	F_1 (N)	P_2 (kN/m^2)	F_2 (N)	F_{ave} (N)	$F/(0.5\gamma A H_{max})$	$gT^2/2\pi L$
0.85	1.047	6.004	0.696	0.890	0.980	30.271	0.872	26.948	28.609	0.2607889	1.077923
0.91	1.113	5.644	0.714	0.898	0.906	27.997	0.814	25.154	26.576	0.2217201	1.161341
0.95	1.037	6.059	0.694	0.889	0.779	24.078	0.693	21.405	22.742	0.1651401	1.358806
1.10	1.249	5.032	0.750	0.913	0.754	23.290	0.688	21.264	22.277	0.1514399	1.513043
1.14	1.020	6.162	0.689	0.887	0.761	23.503	0.674	20.840	22.171	0.1575705	1.989878
1.15	1.475	4.259	0.806	0.932	0.725	22.394	0.675	20.868	21.631	0.1370234	1.399602
1.20	1.429	4.396	0.795	0.929	0.633	19.546	0.587	18.149	18.848	0.1048307	1.573108
1.25	1.632	3.851	0.839	0.942	0.810	25.038	0.763	23.581	24.309	0.1690102	1.495095
1.35	1.703	3.690	0.853	0.946	0.671	20.742	0.635	19.616	20.179	0.115353	1.671162
1.50	1.462	4.297	0.803	0.931	0.371	11.451	0.345	10.660	11.056	0.0358699	2.402508

At water depth, d = 0.3m

Wave Period	L (m)	k	α_1	α_2	P_1 (kN/m^2)	F_1 (N)	P_2 (kN/m^2)	F_2 (N)	F_{ave} (N)	$F/(0.5\gamma A H_{max})$	$gT^2/2\pi L$
0.85	1.040	6.044	0.619	0.889	1.268	39.169	1.126	34.804	36.986	0.4571512	1.085182
0.91	1.289	4.873	0.650	0.909	1.066	32.933	0.968	29.920	31.427	0.3204585	1.002731
0.95	1.337	4.698	0.657	0.912	1.217	37.591	1.109	34.282	35.936	0.4164103	1.053598
1.10	1.420	4.424	0.670	0.918	1.194	36.884	1.095	33.845	35.364	0.3988579	1.330318
1.14	1.493	4.210	0.683	0.922	1.071	33.084	0.987	30.510	31.797	0.3193213	1.359517
1.15	1.453	4.324	0.676	0.920	0.902	27.881	0.830	25.643	26.762	0.2274138	1.421082
1.20	1.608	3.908	0.703	0.929	0.850	26.270	0.790	24.404	25.337	0.1996407	1.398536
1.25	1.452	4.328	0.676	0.920	0.738	22.810	0.679	20.977	21.893	0.1522132	1.680244
1.35	1.257	4.998	0.645	0.906	0.725	22.390	0.657	20.288	21.339	0.1483593	2.263533
1.50	2.125	2.956	0.792	0.952	0.860	26.562	0.818	25.279	25.920	0.1962305	1.652841

'ABULATED DATA FOR CALIBRATION (FORCE VS. DISPLACEMENT) GRAPH

or water depth, d = 20cm

Load (N)	Displacement (cm)
0.0	0.0
0.2	0.2
0.6	1.0
1.2	1.9
1.8	2.6
2.0	2.9
2.4	3.6
2.8	4.2
3.2	4.8
3.6	5.4
4.0	6.1
4.4	6.7
4.8	7.4
5.0	7.7
5.2	8.2

ɔr water depth, d = 30cm

Load (N)	Displacement (cm)
0.0	0.0
0.2	1.5
0.6	3.5
1.2	5.8
1.8	7.9
2.0	8.4
2.4	10.0
2.8	11.3
3.2	12.7
3.6	13.8
4.0	15.1
4.4	16.2

Model M-1, at water depth, d = 0.2m

Wave Period, T(s)	Displacement (cm)				Force (N)	Wave Height, H(cm)	Area (cm2)	Wave Length, L(cm)	$F/(0.5\gamma AH_{max})$	$gT^2/2\pi L$
	1	2	3	Average						
0.85	4.00	5.00	4.25	4.42	5.7717	7.8	180.00	104.65	0.822179	1.077923
0.91	5.00	5.00	5.75	5.25	6.8607	7.2	180.00	111.33	1.058750	1.161341
0.95	4.25	6.00	5.25	5.17	6.7518	6.8	180.00	103.70	1.103235	1.358806
1.10	5.25	5.25	5.25	5.25	6.8607	6.4	180.00	124.86	1.191094	1.513043
1.14	5.50	5.50	5.00	5.33	6.9696	6.0	180.00	101.97	1.290667	1.989878
1.15	4.50	4.75	4.75	4.67	6.0984	5.5	180.00	147.53	1.232000	1.399602
1.20	4.50	5.25	5.50	5.08	6.6429	5.5	180.00	142.92	1.342000	1.573108
1.25	6.25	5.25	5.50	5.67	7.4052	4.3	180.00	163.17	1.913488	1.495095
1.35	6.75	7.50	6.50	6.92	9.0387	4.1	180.00	170.27	2.449512	1.671162
1.50	5.50	5.25	5.00	5.25	6.8607	3.1	180.00	146.22	2.459032	1.883876

At water depth, d = 0.3m

Wave Period, T(s)	Displacement (cm)				Force (N)	Wave Height, H(cm)	Area (cm2)	Wave Length, L(m)	$F/(0.5\gamma AH_{max})$	$gT^2/2\pi L$
	1	2	3	Average						
0.85	5.00	4.25	4.50	4.58	2.0037	10.4	180.00	103.95	0.214065	1.085182
0.91	4.50	5.50	5.75	5.25	2.3729	9.3	180.00	128.94	0.283495	1.002731
0.95	5.00	5.25	4.25	4.83	2.1421	9.0	180.00	133.74	0.264457	1.053598
1.10	4.00	5.00	6.50	5.17	2.3267	8.1	180.00	142.01	0.319163	1.330318
1.14	4.75	4.25	4.75	4.58	2.0037	7.3	180.00	149.25	0.304970	1.359517
1.15	5.25	5.00	5.75	5.33	2.4190	5.0	180.00	145.30	0.537556	1.421082
1.20	4.75	5.00	4.75	4.83	2.1421	6.3	180.00	160.76	0.377795	1.398536
1.25	6.75	6.50	6.00	6.42	3.0190	4.3	180.00	145.19	0.780090	1.680244
1.35	7.25	6.75	7.00	7.00	3.3420	5.8	180.00	125.71	0.640230	2.263533
1.50	6.00	7.25	5.25	6.17	2.8805	2.1	180.00	212.54	1.524074	1.712940

Wave Period, T(s)	Displacement (cm)				Force (N)	Wave Height, H(cm)	Area (cm2)	Wave Length, L(cm)	$F/(0.5\gamma AH_{max})$	$gT^2/2\pi L$
	1	2	3	Average						
0.85	6.00	4.75	5.25	5.33	6.9696	7.1	195.00	104.65	1.006804	1.077923
0.91	5.50	4.50	4.25	4.75	6.2073	6.5	195.00	111.33	0.979456	1.161341
0.95	3.50	4.50	4.00	4.00	5.2272	6.1	195.00	103.70	0.878890	1.358806
1.10	4.00	5.25	4.50	4.58	5.9895	5.5	195.00	124.86	1.116923	1.513043
1.14	3.75	5.25	4.00	4.33	5.6628	4.2	195.00	101.97	1.382857	1.989878
1.15	5.00	5.00	5.00	5.00	6.5340	3.3	195.00	147.53	2.030769	1.399602
1.20	4.50	4.25	4.00	4.25	5.5539	7.5	195.00	142.92	0.759508	1.573108
1.25	5.25	6.00	4.25	5.17	6.7518	4.8	195.00	163.17	1.442692	1.495095
1.35	6.25	6.00	6.75	6.33	8.2764	4.8	195.00	170.27	1.768462	1.671162
1.50	4.25	4.00	5.50	4.58	5.9895	3.1	195.00	146.22	1.981638	1.883876

At water depth, d = 0.3m

Wave Period, T(s)	Displacement (cm)				Force (N)	Wave Height, H(cm)	Area (m2)	Wave Length, L(m)	$F/(0.5\gamma AH_{max})$	$gT^2/2\pi L$
	1	2	3	Average						
0.85	7.25	5.50	5.75	6.17	2.8805	9.5	195.00	103.95	0.310985	1.085182
0.91	6.50	6.25	5.50	6.08	2.8344	10.0	195.00	128.94	0.290703	1.002731
0.95	5.00	6.25	6.00	5.75	2.6498	8.2	195.00	133.74	0.331426	1.053598
1.10	4.00	5.25	6.50	5.25	2.3729	7.5	195.00	142.01	0.324492	1.330318
1.14	6.75	8.25	5.25	6.75	3.2036	7.5	195.00	149.25	0.438092	1.359517
1.15	6.25	7.50	7.75	7.17	3.4343	8.0	195.00	145.30	0.440295	1.421082
1.20	4.00	5.75	5.25	5.00	2.2344	6.6	195.00	160.76	0.347226	1.398536
1.25	7.00	8.25	6.25	7.17	3.4343	6.0	195.00	145.19	0.587060	1.680244
1.35	9.00	7.75	8.25	8.33	4.0804	7.2	195.00	125.71	0.581254	2.263533
1.50	5.25	6.75	7.00	6.33	2.9728	4.9	195.00	212.54	0.622250	1.712940

Wave Period, T(s)					Force (N)	Height, H(cm)	Area (cm2)	Length, L(cm)	$F/(0.5\gamma AH_{max})$	$gT^2/2\pi L$
	1	2	3	Average						
0.85	3.00	3.50	3.25	3.25	4.2471	7.2	147.00	104.65	0.802551	1.077923
0.91	3.25	3.00	4.00	3.42	4.4649	7.5	147.00	111.33	0.809959	1.161341
0.95	4.25	3.00	3.50	3.58	4.6827	6.6	147.00	103.70	0.965306	1.358806
1.10	3.25	4.50	3.25	3.67	4.7916	7.0	147.00	124.86	0.931312	1.513043
1.14	4.50	3.25	4.00	3.92	5.1183	5.7	147.00	101.97	1.221697	1.989878
1.15	4.50	3.25	5.00	4.25	5.5539	4.5	147.00	147.53	1.679184	1.399602
1.20	3.25	3.75	3.00	3.33	4.3560	6.7	147.00	142.92	0.884557	1.573108
1.25	4.50	5.50	4.75	4.92	6.4251	5.8	147.00	163.17	1.507178	1.495095
1.35	5.00	6.75	6.00	5.92	7.7319	4.8	147.00	170.27	2.191582	1.671162
1.50	4.25	4.25	4.00	4.17	5.4450	4.3	147.00	146.22	1.722829	1.883876

At water depth, d = 0.3m

Wave Period, T(s)	Displacement (cm)				Force (N)	Wave Height, H(cm)	Area (m2)	Wave Length, L(m)	$F/(0.5\gamma AH_{max})$	$gT^2/2\pi L$
	1	2	3	Average						
0.85	6.00	6.50	5.50	6.00	2.7882	9.0	147.00	103.95	0.421497	1.085182
0.91	5.75	7.25	6.25	6.42	3.0190	9.4	147.00	128.94	0.436959	1.002731
0.95	6.00	7.50	6.50	6.67	3.1574	10.2	147.00	133.74	0.421155	1.053598
1.10	6.00	7.75	7.00	6.92	3.2959	10.5	147.00	142.01	0.427062	1.330318
1.14	6.50	6.75	6.75	6.67	3.1574	9.4	147.00	149.25	0.456998	1.359517
1.15	7.25	6.50	7.75	7.17	3.4343	8.8	147.00	145.30	0.530968	1.421082
1.20	5.50	6.25	6.75	6.17	2.8805	7.5	147.00	160.76	0.522540	1.398536
1.25	6.75	7.75	7.75	7.42	3.5728	9.4	147.00	145.19	0.517115	1.680244
1.35	8.25	8.75	8.50	8.50	4.1727	6.7	147.00	125.71	0.847335	2.263533
1.50	6.50	5.50	6.25	6.08	2.8344	6.4	147.00	212.54	0.599729	1.712940

Wave Period, T(s)	1	2	3	Average	Force (N)	Height, H(cm)	Area (cm2)	Length, L(cm)	$F/(0.5\gamma AH_{max})$	$gT^2/2\pi L$
0.85	7.50	7.50	7.00	7.33	9.5832	5.9	309.00	104.65	1.051308	1.077923
0.91	6.00	6.75	6.00	6.25	8.1675	5.4	309.00	111.33	0.978964	1.161341
0.95	5.80	5.75	5.75	5.77	7.5359	4.7	309.00	103.70	1.037786	1.358806
1.10	5.25	5.00	5.20	5.15	6.7300	4.4	309.00	124.86	0.990000	1.513043
1.14	5.75	5.50	5.50	5.58	7.2963	4.6	309.00	101.97	1.026636	1.989878
1.15	5.50	5.90	6.00	5.80	7.5794	4.1	309.00	147.53	1.196533	1.399602
1.20	4.90	5.00	4.50	4.80	6.2726	3.6	309.00	142.92	1.127767	1.573108
1.25	4.75	4.25	4.60	4.53	5.9242	4.5	309.00	163.17	0.852091	1.495095
1.35	3.10	2.80	3.30	3.07	4.0075	3.7	309.00	170.27	0.701044	1.671162
1.50	3.25	3.10	3.20	3.18	4.1600	2.1	309.00	146.22	1.282164	1.883876

At water depth, d = 0.3m

Wave Period, T(s)	Displacement (cm)				Force (N)	Wave Height, H(cm)	Area (m2)	Wave Length, L(m)	$F/(0.5\gamma AH_{max})$	$gT^2/2\pi L$
	1	2	3	Average						
0.85	11.25	11.00	11.25	11.17	5.6495	8.0	309.00	103.95	0.457079	1.085182
0.91	9.50	9.40	9.25	9.38	4.6619	6.6	309.00	128.94	0.457183	1.002731
0.95	7.50	7.75	7.75	7.67	3.7112	7.5	309.00	133.74	0.320276	1.053598
1.10	10.50	11.00	10.75	10.75	5.4188	7.3	309.00	142.01	0.480450	1.330318
1.14	10.50	10.50	10.50	10.50	5.2803	6.5	309.00	149.25	0.525795	1.359517
1.15	7.50	7.75	7.75	7.67	3.7112	5.5	309.00	145.30	0.436740	1.421082
1.20	9.00	10.75	10.25	10.00	5.0034	5.1	309.00	160.76	0.634990	1.398536
1.25	8.50	8.25	8.40	8.38	4.1081	4.5	309.00	145.19	0.590880	1.680244
1.35	6.50	6.75	6.40	6.55	3.0928	3.9	309.00	125.71	0.513284	2.263533
1.50	6.00	6.50	6.20	6.23	2.9174	4.9	309.00	212.54	0.385367	1.712940

Cycle	Displacement (cm)		x/L		Incident Wave Height (cm)	Attenuated Wave Height (cm)	H/d	
	Backward	Forward	Backward	Forward			Incident	Attenuated
1	-2.0	2.6	-0.0117	0.0153	2.8	1.1	0.140	0.055
2	-1.8	2.9	-0.0106	0.0170	2.8	1.0	0.140	0.050
3	-2.2	2.9	-0.0129	0.0170	3.0	1.0	0.150	0.050
4	-2.2	3.3	-0.0129	0.0194	3.0	1.0	0.150	0.050
5	-2.2	3.3	-0.0129	0.0194	3.0	1.1	0.150	0.055
6	-2.3	3.5	-0.0135	0.0206	3.0	1.0	0.150	0.050
7	-2.1	3.3	-0.0123	0.0194	3.0	1.0	0.150	0.050
8	-2.1	2.8	-0.0123	0.0164	3.1	0.9	0.155	0.045
9	-2.2	2.9	-0.0129	0.0170	3.0	0.9	0.150	0.045
10	-2.2	3.1	-0.0129	0.0182	3.0	1.0	0.150	0.050
11	-2.0	4.0	-0.0117	0.0235	3.1	1.0	0.155	0.050
12	-2.0	3.8	-0.0117	0.0223	3.0	1.0	0.150	0.050
13	-2.1	3.4	-0.0123	0.0200	3.0	1.1	0.150	0.055
14	-2.0	3.3	-0.0117	0.0194	3.0	1.1	0.150	0.055
15	-2.0	3.4	-0.0117	0.0200	3.1	1.0	0.155	0.050

Forward Displacement

x_{ave} = 3.233 cm
x_{man} = 4.000 cm
x_{min} = 2.600 cm
SD = 0.365
CV = 0.113

Backward Displacement

x_{ave} = 0.079 cm
x_{man} = 0.155 cm
x_{min} = 0.140 cm
SD = 0.124
CV = 1.565

* SD = Standard Deviation
CV = Coefficient of Variation

Cycle	Backward	Forward	Backward	Forward	Incident Wave Height (cm)	Wave Height (cm)	Incident	Attenuated
1	-4.8	6.1	-0.0382	0.0485	10.1	4.3	0.080	0.034
2	-5.8	3.7	-0.0461	0.0294	8.9	6.3	0.071	0.050
3	-3.2	6.8	-0.0255	0.0541	10.8	5.6	0.086	0.045
4	-5.8	1.5	-0.0461	0.0119	9.1	6.2	0.072	0.049
5	-4.9	10.1	-0.0390	0.0803	11.2	5.8	0.089	0.046
6	-5.6	1.1	-0.0445	0.0088	9.3	6.0	0.074	0.048
7	-4.2	9.6	-0.0334	0.0764	11.6	6.1	0.092	0.049
8	-3.9	0.3	-0.0310	0.0024	11.4	6.7	0.091	0.053
9	-5.7	7.3	-0.0453	0.0581	10.6	5.8	0.084	0.046
10	-4.0	0.5	-0.0318	0.0040	10.3	6.3	0.082	0.050
11	-4.0	7.9	-0.0318	0.0628	9.7	6.1	0.077	0.049
12	-4.9	0.0	-0.0390	0.0000	10.1	5.5	0.080	0.044
13	-2.4	10.8	-0.0191	0.0859	9.2	5.8	0.073	0.046
14	-6.6	1.1	-0.0525	0.0088	10.2	5.8	0.081	0.046
15	-2.1	13.4	-0.0167	0.1066	8.9	5.9	0.071	0.047

Forward Displacement

x_{ave} = 5.347 cm
x_{man} = 13.400 cm
x_{min} = 0.000 cm
SD = 4.316
CV = 0.807

Backward Displacement

x_{ave} = -4.527 cm
x_{man} = -2.1 cm
x_{min} = -6.6 cm
SD = 1.2556
CV = -0.277

* SD = Standard Deviation
CV = Coefficient of Variation

Cycle	Backward	Forward	Backward	Forward	Incident Wave Height (cm)	Wave Height (cm)	Incident	Attenuated
1	-1.1	2.4	-0.0065	0.0141	1.4	1.8	0.070	0.090
2	-0.7	1.8	-0.0041	0.0106	2.2	1.0	0.110	0.050
3	-0.8	2.2	-0.0047	0.0129	2.4	1.1	0.120	0.055
4	-1.1	2.5	-0.0065	0.0147	1.8	1.1	0.090	0.055
5	-0.3	1.8	-0.0018	0.0106	1.9	1.0	0.095	0.050
6	-0.6	1.7	-0.0035	0.0100	1.5	1.0	0.075	0.050
7	-0.3	1.5	-0.0018	0.0088	1.6	1.1	0.080	0.055
8	-0.9	1.8	-0.0053	0.0106	1.5	1.1	0.075	0.055
9	-0.5	1.8	-0.0029	0.0106	1.6	1.1	0.080	0.055
10	-0.6	2.0	-0.0035	0.0117	1.6	1.1	0.080	0.055
11	-1.0	2.1	-0.0059	0.0123	1.6	1.0	0.080	0.050
12	-0.5	1.9	-0.0029	0.0112	1.3	1.0	0.065	0.050
13	-0.9	1.8	-0.0053	0.0106	1.0	1.1	0.050	0.055
14	0.0	1.8	0.0000	0.0106	1.4	1.0	0.070	0.050
15	0.0	1.9	0.0000	0.0112	1.5	1.1	0.075	0.055

Forward Displacement

$x_{ave} =$ 1.933 cm
$x_{man} =$ 2.500 cm
$x_{min} =$ 1.500 cm
SD = 0.257
CV = 0.133

Backward Displacement

$x_{ave} =$ 0.048 cm
$x_{man} =$ 0.120 cm
$x_{min} =$ 0.070 cm
SD = 0.347
CV = 7.275

* SD = Standard Deviation
CV = Coefficient of Variation

Cycle	Backward	Forward	Backward	Forward	Incident Wave Height (cm)	Wave Height (cm)	Incident	Attenuated
1	-3.6	5.0	-0.0286	0.0398	5.8	4.1	0.193	0.137
2	-5.1	5.5	-0.0406	0.0438	6.0	3.8	0.200	0.127
3	-2.8	4.2	-0.0223	0.0334	6.1	4.1	0.203	0.137
4	-3.7	5.5	-0.0294	0.0438	7.1	2.4	0.237	0.080
5	-3.8	6.8	-0.0302	0.0541	6.3	2.2	0.210	0.073
6	-4.1	5.1	-0.0326	0.0406	6.3	3.1	0.210	0.103
7	-4.2	4.8	-0.0334	0.0382	7.8	4.3	0.260	0.143
8	-2.2	5.2	-0.0175	0.0414	7.2	5.8	0.240	0.193
9	-3.2	2.6	-0.0255	0.0207	7.1	6.0	0.237	0.200
10	-2.6	7.6	-0.0207	0.0605	7.2	6.0	0.240	0.200
11	-2.9	9.2	-0.0231	0.0732	6.1	6.1	0.203	0.203
12	-3.6	0.3	-0.0286	0.0024	6.4	4.2	0.213	0.140
13	-2.7	9.6	-0.0215	0.0764	5.6	3.7	0.187	0.123
14	-7.3	5.2	-0.0581	0.0414	8.1	3.8	0.270	0.127
15	-4.8	8.7	-0.0382	0.0692	6.3	4.1	0.210	0.137

Forward Displacement

x_{ave} = 5.687 cm
x_{man} = 9.600 cm
x_{min} = 0.300 cm
SD = 2.374
CV = 0.417

Backward Displacement

x_{ave} = 0.117 cm
x_{man} = 0.260 cm
x_{min} = 0.193 cm
SD = 1.229
CV = 10.513

* SD = Standard Deviation
CV = Coefficient of Variation

	Backward	Forward	Backward	Forward	Height (cm)	Wave Height (cm)	Incident	Attenuated
1	-2.2	1.5	-0.0129	0.0088	4.2	3.2	0.210	0.160
2	-2.5	1.5	-0.0147	0.0088	4.1	3.1	0.205	0.155
3	-2.4	1.5	-0.0141	0.0088	4.1	3.1	0.205	0.155
4	-2	1.5	-0.0117	0.0088	4.3	3.1	0.215	0.155
5	-2.3	1.2	-0.0135	0.0070	4.1	3.2	0.205	0.160
6	-2.2	1.4	-0.0129	0.0082	4.3	3.2	0.215	0.160
7	-2.2	1.5	-0.0129	0.0088	4.5	3.1	0.225	0.155
8	-2.6	1.2	-0.0153	0.0070	4.4	3.3	0.220	0.165
9	-2.9	1.6	-0.0170	0.0094	4.3	3.1	0.215	0.155
10	-2.9	1.6	-0.0170	0.0094	4.1	3.1	0.205	0.155
11	-2.6	1.2	-0.0153	0.0070	4.2	3.1	0.210	0.155
12	-2.7	1.2	-0.0159	0.0070	4.2	3.2	0.210	0.160
13	-2.9	1.6	-0.0170	0.0094	4.1	3.1	0.205	0.155
14	-2.7	1.4	-0.0159	0.0082	4	3.3	0.200	0.165
15	-2.6	1.6	-0.0153	0.0094	4	3.2	0.200	0.160

Forward Displacement

x_{ave} = 1.433 cm
x_{man} = 1.600 cm
x_{min} = 1.200 cm
SD = 0.153
CV = 0.107

Backward Displacement

x_{ave} = 0.113 cm
x_{man} = 0.225 cm
x_{min} = 0.205 cm
SD = 0.278
CV = 2.451

* SD = Standard Deviation
CV = Coefficient of Variation

Cycle	Backward	Forward	Backward	Forward	Incident Wave Height (cm)	Wave Height (cm)	Incident	Attenuated
1	-5.2	6.1	-0.0414	0.0485	9.3	5.6	0.310	0.187
2	-5.0	6.5	-0.0398	0.0517	9.3	5.9	0.310	0.197
3	-5.2	4.5	-0.0414	0.0358	9.1	7.3	0.303	0.243
4	-5.0	6.2	-0.0398	0.0493	9	4.8	0.300	0.160
5	-6.5	5.9	-0.0517	0.0469	9.5	6.3	0.317	0.210
6	-4.8	3.1	-0.0382	0.0247	10	5.1	0.333	0.170
7	-5.6	6.1	-0.0445	0.0485	8.2	5.1	0.273	0.170
8	-4.3	3.5	-0.0342	0.0278	12	4.3	0.400	0.143
9	-5.0	5.6	-0.0398	0.0445	8.3	5.3	0.277	0.177
10	-5.7	2.7	-0.0453	0.0215	12.4	4.2	0.413	0.140
11	-5.6	8.1	-0.0445	0.0644	9.6	5.2	0.320	0.173
12	-6.8	4.3	-0.0541	0.0342	13	5.7	0.433	0.190
13	-7.0	7	-0.0557	0.0557	9.4	6.1	0.313	0.203
14	-5.6	6.8	-0.0445	0.0541	7.5	6.1	0.250	0.203
15	-6.0	7.8	-0.0477	0.0620	9.8	6.2	0.327	0.207

Forward Displacement

x_{ave} = 5.613 cm
x_{man} = 8.100 cm
x_{min} = 2.700 cm
SD = 1.596
CV = 0.284

Backward Displacement

x_{ave} = 0.170 cm
x_{man} = 0.400 cm
x_{min} = 0.273 cm
SD = 0.734
CV = 4.322

* SD = Standard Deviation
CV = Coefficient of Variation

Cycle	Backward	Forward	Backward	Forward	Incident Wave Height (cm)	Wave Height (cm)	Incident	Attenuated
1	-2.2	1.9	-0.0129	0.0112	2.2	1.9	0.110	0.095
2	-2.5	2.5	-0.0147	0.0147	2.5	1.8	0.125	0.090
3	-1.9	2.7	-0.0112	0.0159	3.0	1.8	0.150	0.090
4	-2.9	2.6	-0.0170	0.0153	3.1	1.9	0.155	0.095
5	-2.2	2.6	-0.0129	0.0153	2.9	1.8	0.145	0.090
6	-2.2	2.2	-0.0129	0.0129	3.0	1.8	0.150	0.090
7	-1.7	2.6	-0.0100	0.0153	2.9	1.7	0.145	0.085
8	-1.8	2.1	-0.0106	0.0123	3.1	1.7	0.155	0.085
9	-1.5	1.8	-0.0088	0.0106	3.0	1.8	0.150	0.090
10	-1.4	1.9	-0.0082	0.0112	2.7	1.7	0.135	0.085
11	-1.2	1.1	-0.0070	0.0065	2.7	1.9	0.135	0.095
12	-1.2	1.7	-0.0070	0.0100	2.9	2.1	0.145	0.105
13	-1.3	1.3	-0.0076	0.0076	2.4	2.0	0.120	0.100
14	-1.1	1.3	-0.0065	0.0076	2.3	2.0	0.115	0.100
15	-1.2	1.5	-0.0070	0.0088	2.5	2.0	0.125	0.100

Forward Displacement

x_{ave} = 1.987 cm

x_{man} = 2.700 cm

x_{min} = 1.100 cm

SD = 0.153

CV = 0.077

Backward Displacement

x_{ave} = 0.076 cm

x_{man} = 0.155 cm

x_{min} = 0.110 cm

SD = 0.278

CV = 3.671

* SD = Standard Deviation

CV = Coefficient of Variation

Cycle	Backward	Forward	Backward	Forward	Incident Wave Height (cm)	Wave Height (cm)	Incident	Attenuated
1	-2.3	7.2	-0.0183	0.0573	6.2	3.8	0.207	0.127
2	-2.5	7.3	-0.0199	0.0581	5.8	3.8	0.193	0.127
3	-1.7	8.1	-0.0135	0.0644	6.3	3.9	0.210	0.130
4	-3.1	6.4	-0.0247	0.0509	6.2	3.9	0.207	0.130
5	-2.4	8.0	-0.0191	0.0636	6.0	4.0	0.200	0.133
6	-2.6	6.5	-0.0207	0.0517	6.0	4.1	0.200	0.137
7	-2.7	7.0	-0.0215	0.0557	5.9	4.2	0.197	0.140
8	-1.8	7.9	-0.0143	0.0628	5.8	4.2	0.193	0.140
9	-2.0	6.7	-0.0159	0.0533	5.9	4.2	0.197	0.140
10	-2.3	7.2	-0.0183	0.0573	6.0	4.0	0.200	0.133
11	-2.4	7.5	-0.0191	0.0597	6.0	4.0	0.200	0.133
12	-2.4	6.4	-0.0191	0.0509	6.2	4.0	0.207	0.133
13	-2.6	7.3	-0.0207	0.0581	6.2	4.0	0.207	0.133
14	-2.9	7.0	-0.0231	0.0557	6.2	4.0	0.207	0.133
15	-1.9	7.8	-0.0151	0.0620	6.2	4.0	0.207	0.133

Forward Displacement

x_{ave} = 7.220 cm
x_{man} = 8.100 cm
x_{min} = 6.400 cm
SD = 1.596
CV = 0.221

Backward Displacement

x_{ave} = 0.107 cm
x_{man} = 0.210 cm
x_{min} = 0.193 cm
SD = 0.734
CV = 6.851

* SD = Standard Deviation
CV = Coefficient of Variation

Cycle	d = 20cm				d = 30cm				d = 20cm				d = 30cm			
	% Reflected	% Transmitted	% Absorbed	% Dissipated	% Reflected	% Transmitted	% Absorbed	% Dissipated	% Reflected	% Transmitted	% Absorbed	% Dissipated	% Reflected	% Transmitted	% Absorbed	% Dissipated
1	29.09	39.29	9.19	22.43	23.89	42.57	3.89	29.65	23.31	128.57	31.32	-83.20	27.03	70.69	7.92	-5.64
2	29.09	35.71	11.43	23.77	23.89	70.79	1.84	3.48	23.31	45.45	7.13	24.11	27.03	63.33	8.95	0.69
3	29.09	33.33	9.96	27.62	23.89	51.85	4.22	20.04	23.31	45.83	8.95	21.91	27.03	67.21	5.05	0.71
4	29.09	36.67	12.89	21.35	23.89	68.13	0.29	7.69	23.31	61.11	20.56	-4.98	27.03	33.80	6.39	32.78
5	29.09	33.33	12.89	24.69	23.89	51.79	8.67	15.65	23.31	52.63	9.56	14.50	27.03	34.92	12.42	25.63
6	29.09	33.33	14.51	23.07	23.89	64.52	0.15	11.44	23.31	66.67	13.69	-3.67	27.03	49.21	6.98	16.78
7	29.09	29.03	12.89	28.99	23.89	52.59	7.30	16.22	23.31	68.75	9.37	-1.43	27.03	55.13	4.04	13.80
8	29.09	30.00	8.69	32.22	23.89	58.77	0.07	17.27	23.31	73.33	15.35	-11.99	27.03	80.56	5.56	-13.15
9	29.09	33.33	9.69	27.89	23.89	54.72	5.05	16.34	23.31	68.75	13.49	-5.55	27.03	84.51	1.43	-12.97
10	29.09	32.26	11.38	27.27	23.89	61.17	0.25	14.69	23.31	68.75	16.65	-8.71	27.03	83.33	11.87	-22.23
11	29.09	33.33	17.74	19.84	23.89	62.89	70.69	-57.47	23.31	62.50	18.36	-4.17	27.03	100.00	24.24	-51.27
12	29.09	36.67	17.10	17.14	23.89	54.46	0.00	21.65	23.31	76.92	22.76	-22.99	27.03	65.63	0.23	7.11
13	29.09	36.67	13.69	20.55	23.89	63.04	14.69	-1.62	23.31	110.00	34.53	-67.84	27.03	66.07	31.32	-24.42
14	29.09	36.67	12.89	21.35	23.89	56.86	0.12	19.13	23.31	71.43	17.62	-12.36	27.03	46.91	4.39	21.67
15	29.09	32.26	12.82	25.83	23.89	66.29	24.16	-14.34	23.31	73.33	17.10	-13.74	27.03	65.08	20.32	-12.43
Ave	**29.09**	**34.13**	**12.52**	**24.27**	**23.89**	**58.70**	**9.43**	**7.99**	**23.31**	**71.60**	**17.10**	**-12.01**	**27.03**	**64.43**	**10.07**	**-1.53**

Cycle	M3								M4							
	d = 20cm				d = 30cm				d = 20cm				d = 30cm			
	% Reflected	% Transmitted	% Absorbed	% Dissipated	% Reflected	% Transmitted	% Absorbed	% Dissipated	% Reflected	% Transmitted	% Absorbed	% Dissipated	% Reflected	% Transmitted	% Absorbed	% Dissipated
1	16.00	76.19	1.36	6.45	19.27	60.22	4.58	15.93	11.11	86.36	7.95	-5.42	28.30	61.29	14.37	-3.96
2	16.00	75.61	1.43	6.96	19.27	63.44	5.21	12.08	11.11	72.00	10.66	6.23	28.30	65.52	16.88	-10.70
3	16.00	75.61	1.43	6.96	19.27	80.22	2.61	-2.10	11.11	60.00	8.63	20.26	28.30	61.90	17.62	-7.82
4	16.00	72.09	1.30	10.61	19.27	53.33	5.06	22.34	11.11	61.29	7.50	20.10	28.30	62.90	11.36	-2.56
5	16.00	78.05	0.91	5.04	19.27	66.32	4.11	10.30	11.11	62.07	8.57	18.25	28.30	66.67	18.95	-13.92
6	16.00	74.42	1.13	8.45	19.27	51.00	1.02	28.71	11.11	60.00	5.73	23.16	28.30	68.33	12.51	-9.14
7	16.00	68.89	1.18	13.93	19.27	62.20	5.90	12.63	11.11	58.62	8.57	21.70	28.30	71.19	15.00	-14.49
8	16.00	75.00	0.79	8.21	19.27	35.83	0.91	43.99	11.11	54.84	4.89	29.16	28.30	72.41	19.77	-20.48
9	16.00	72.09	1.48	10.43	19.27	63.86	4.85	12.02	11.11	60.00	3.84	25.05	28.30	71.19	13.74	-13.23
10	16.00	75.61	1.62	6.77	19.27	33.87	0.51	46.35	11.11	62.96	5.28	20.65	28.30	66.67	15.35	-10.32
11	16.00	73.81	0.87	9.32	19.27	54.17	7.69	18.87	11.11	70.37	1.77	16.75	28.30	66.67	16.65	-11.62
12	16.00	76.19	0.87	6.94	19.27	43.85	1.17	35.71	11.11	72.41	3.66	12.82	28.30	64.52	11.36	-4.18
13	16.00	75.61	1.62	6.77	19.27	64.89	5.91	9.93	11.11	83.33	3.13	2.43	28.30	64.52	14.77	-7.59
14	16.00	82.50	1.31	0.19	19.27	81.33	8.76	-9.36	11.11	86.96	3.40	-1.47	28.30	64.52	13.58	-6.40
15	16.00	80.00	1.71	2.29	19.27	63.27	6.75	10.71	11.11	80.00	3.84	5.05	28.30	64.52	16.87	-9.69
Ave	**16.00**	**75.44**	**1.27**	**7.29**	**19.27**	**58.52**	**4.34**	**17.87**	**11.11**	**68.75**	**5.83**	**14.31**	**28.30**	**66.19**	**15.25**	**-9.74**